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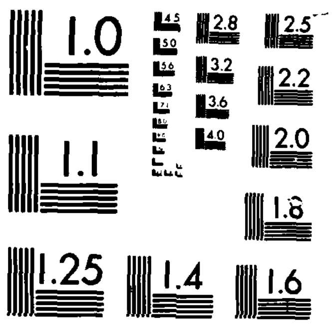

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US Army Corps of Engineers
Los Angeles District

West Magnesia Canyon Channel
City of Rancho Mirage
Riverside County, California

AD-A150 305

Detailed Project Report

RANCHO MIRAGE

flood control

Technical Appendixes

FINAL

December 1983

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APPENDIXES

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U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

SEPTEMBER 1983

MAGNESIA SPRING CANYON
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APPENDIX A

HYDROLOGY

I. INTRODUCTION

1.01. PURPOSE AND SCOPE. This report presents hydrology in support of Detailed Project Report (DPR) studies for Magnesia Spring Canyon, Riverside County, California. The report has four major objectives: (a) to present the basic meteorologic and hydrologic characteristics of the study area; (b) to outline the methods and techniques used to model the runoff process and to determine discharge frequency relationships; (c) to present standard project flood and discharge frequency values under preproject and project conditions; and (d) to present probable maximum flood and debris production estimates for the proposed debris basin. The general location of the study area is shown on plate A-1; plate A-2 shows drainage area boundaries. Tables A-1 and A-2, and plates A-9 and A-10, give peak discharge values for preproject and project conditions. Subarea characteristics are given in table A-3.

1.02. PREVIOUS REPORTS. Prior hydrology for the Magnesia Spring Canyon basin was presented in the report entitled "Whitewater River Basin, Feasibility Report for Flood Control," dated May 1980 (ref. 1). The current report expands the scope of the previous study. Other references with material of hydrologic importance for the study area are listed in the bibliography.

1.03. PROPOSED PLAN OF IMPROVEMENT. The proposed improvement consists of a combination of flood control channel and debris basin. The channel would extend from the Whitewater River to approximately 1.3 miles upstream, where the construction of the debris basin is proposed. The level of protection provided by the channel would be standard project flood, and the debris basin would be designed to contain the debris production of a single, large flood event.

1.04. COORDINATION WITH OTHER AGENCIES. A draft of this report was forwarded for review and comment to Riverside County Sanitation and Flood Control District, Coachella Valley Water District (CVWD), and Bechtel Corporation, consultants to CVWD.

II. GENERAL DESCRIPTION OF THE DRAINAGE AREA

2.01. BASIN DESCRIPTION. Magnesia Spring Canyon, a tributary of the Whitewater River, originates in the lower San Jacinto Mountains in Riverside County (see plate A-1). The stream flows in a northeast direction. At elevation 600 feet, the stream enters an alluvial fan area where the flow path becomes undefined depending on the magnitude of flows. Low flows are directed by existing levees to the West Magnesia Channel, which runs from elevation 480 feet to the Whitewater River. Large floodflows would cause the levees to fail and flood the Rancho Mirage community. The 5 square mile portion of the basin above elevation 500 feet is about 2.5 miles long, with an average width of about 2 miles (see plate A-2). Elevations range from about 220 feet at the Whitewater River to 2,975 feet in the higher peaks, with an average elevation of 1,500 feet. The average gradient upstream of the fan area is about 600 feet per mile, decreasing to about 190 feet per mile downstream.

2.02. GEOLOGY AND SOILS. Magnesia Spring Canyon is typical of the mountain canyons at the toe of the San Jacinto Mountains, containing steep walls bordering a relatively flat floor. The walls contain mostly hard and relatively massive metamorphic rocks to an elevation of about 1,600 feet. Above that elevation, the slopes become markedly flatter and are covered with a thin veneer of residual sandy soil. The canyon floor is the head of a large alluvial fan up to about elevation 600 feet. The fan contains mostly clean sand, with some gravel, cobbles, and boulders to 3-foot diameter. Scattered remnants of former fans exist as terraces about 40 feet above the current fan.

2.03. VEGETATION. Typical desert vegetation such as scattered creosote bush, ocotilla, paloverde, ironwood, and cactus grow in the lower slopes of the San

Jacinto Mountains. In the flat areas, the watercourses are thinly lined with mesquite, ranging from stunted shrubs to small trees.

2.04. LAND USE. No future development is expected to occur within the Magnesia Spring Canyon basin that might alter the runoff characteristics of the watershed. Although some development would occur on the cone with the proposed project, the additional runoff would not contribute to West Magnesia Channel flows.

2.05. HYDROMETEOROLOGICAL CHARACTERISTICS.

a. The study area is characterized by a subtropical desert climate, with hot, dry summers and mild winters. The desert floor is one of the hottest areas in North America during the months of June through August, with daily maximum temperatures of 110 to 115 degrees Fahrenheit very common and with all-time highs of around 125 degrees Fahrenheit. During colder winter nights, the temperature can occasionally drop below freezing. The mountain slopes of the study area are generally cooler than the desert floor, especially during daytime hours, with a temperature decrease of about 5 degrees Fahrenheit per 1,000 feet of elevation. Prevailing winds in the region are generally from the northwest and are usually strongest during spring and summer.

b. The mean annual precipitation is very low on the desert floor, with only about 4.5 inches in Rancho Mirage along Highway 111. This increases to about 6 inches in the highest portions of the study area. Most of the precipitation falls during the cooler months, November through March, but high-intensity thunderstorms and even tropical storms can occasionally occur between mid-summer and early fall.

c. Three types of storms can produce precipitation in the study area: general winter storms, general summer storms, and local storms. A brief description of each storm type is given in the following subparagraphs.

(1) General winter storms usually occur during the period from November through March. They originate over the Pacific Ocean and move across the basin generally from west to east. They normally last from one-half day to several days and are accompanied by widespread precipitation. Those storms which move into the area from out of the subtropical Pacific southwest of southern California are usually heavier than those which originate in the Gulf of Alaska and approach the region from out of the northwest.

(2) General summer storms are quite rare in the study area and are generally limited to the period early August through early October. They normally move into the region from out of the south or southeast and are often associated with the remnants of a tropical hurricane from off the west coast of Mexico. In a general summer storm, there is often widespread moderate precipitation for durations up to 24 hours, with showers lasting up to 3 days. Some heavy general precipitation and very heavy local thunderstorms are sometimes imbedded.

(3) Local storms can occur at any time of the year, either during general storms or as isolated phenomena. The most frequent and potentially heaviest local storms usually occur in the study area from July through September, but fairly heavy local storms can also occur from December through March. These local storms cover comparatively small areas and frequently result in high-intensity precipitation of short duration. The storms are usually accompanied

by considerable lighting and thunder and are often accompanied by strong, gusty winds and/or hail.

2.06. RUNOFF CHARACTERISTICS. Except for periods during and immediately following rainstorms, there is little or no streamflow. Climatic and drainage area characteristics are not conducive to continuous runoff. During the larger storms, especially those occurring soon after other storms, the streamflow increases rapidly in response to effective precipitation. Floods are of the flashy type, having sharp peaks and short durations. Large floods transport moderate quantities of debris that is usually deposited at the base of the canyon. Considerable percolation would occur on the debris cone during large floods. Baseflow is considered negligible. Snowmelt is not a contributing factor to runoff.

2.07. EXISTING AND PROPOSED STRUCTURES AFFECTING RUNOFF.

a. West Magnesia Channel, a combination of levee and channel approximately 1.3 miles long, was built along the west side of Rancho Mirage by local agencies. The effectiveness of the upstream levee is questionable, however. During the September 1976 flood, an estimated peak discharge of 800 cfs broke through the levee, as did the much larger flood of July 1979, when the peak discharge was estimated to be between 5,000 and 7,000 cfs.

b. The Coachella Valley Water District is proposing the contruction of an East Magnesia Channel on the east side of Rancho Mirage that would provide SPF level of protection from runoff coming from the adjacent foothills. This project was considered in place for existing conditions.

III. STORMS AND FLOODS OF RECORD

3.01. GENERAL. Little information is available pertaining to floods in th Magnesia Spring Canyon basin. The following paragraphs give a brief description of the storm of 24 September 1939, which was used to develop th standard project flood, and the events of 9-11 September 1976 and 20 July 1 Historical accounts of other storms and floods that have occurred in the Whitewater River basin are given in reference 1.

3.02. STORM OF 24 SEPTEMBER 1939. At Indio, in a thunderstorm preceding t occurrence of a tropical st.m from off the west coast of Mexico, 6.45 inch fell in a period of 6 hours. Short-time intensities during this burst of precipitation, as noted by the observer at Indio, are shown in table A-5. No estimates of runoff during this thunderstorm are available. The total precipitation in the Whitewater River basin from the tropical storm varied from 9.65 inches at Raywood Flat in the San Bernardino Mountains to 1.51 inches at Palm Springs.

3.03. STORM AND FLOOD OF 9-11 SEPTEMBER 1976.

a. During the period 9-11 September 1976, Tropical Storm Kathleen was steered by atmospheric currents northward from off the west coast of Mexicc and into the Imperial and Coachella Valleys of California. The passage of this storm generated very heavy general rainfall over the mountains and deserts of San Bernardino, Riverside, and Imperial Counties. Total storm precipitation in the Rancho Mirage and Palm Desert area was around 3 inches (2.95" at Cathedral City Road Department and 3.32" at Palm Desert Fire Station), with higher totals in the foothills and up to 14 inches in the hi

mountains. The upper portions of Deep Canyon (a neighboring basin southeast of Magnesia Spring Canyon) received up to 8 inches of rain. Most of the precipitation in this storm fell during the morning of 10 September, and the highest intensities occurred during the late morning, when rates of more than 1 inch in 1 hour were recorded.

b. Despite the fact that the ground was generally dry at the beginning of the storm, the amounts and intensities of rainfall during the earlier hours of the storm easily saturated the ground, so that a large portion of the heavy late-morning rain of 10 September ran off. The peak discharge at the mouth of Magnesia Spring Canyon was estimated by the Corps of Engineers to be only 800 cfs, but flow rates on some neighboring streams were much higher. At the USGS stream gage in Deep Canyon (drainage area = 30.6 sq. mi.), the peak discharge was 7,100 cfs; on Dead Indian Creek near Palm Desert (located between Deep Canyon and Magnesia Spring Canyon, and having a drainage area of 9.02 sq. mi.), the Corps of Engineers estimated a peak of 8,900 cfs.

3.04. STORM AND FLOOD OF 20 JULY 1979. During the early hours of 20 July 1979, an intense local thunderstorm broke over the foothill areas from Palm Springs to La Quinta. The center of the storm was in the southwestern portions of Rancho Mirage and Cathedral City and in the hills above these communities. Although the very heaviest rainfall in this storm might not have been measured, the gage at the Cathedral City Fire Station recorded a maximum of 1.37 inches in 30 minutes, 2.24 inches in 1 hour, 2.92 inches in 2 hours, 3.19 inches in 3 hours, and 3.68 inches in 6 hours. Because of this extremely high-intensity rainfall over the steep foothill terrain above Rancho Mirage and Cathedral City, very heavy runoff developed in a matter of minutes, and

severe flash flooding occurred in these communities. Peak discharges at the mouth of the Magnesia Spring Canyon and on a small tributary were estimated by the Corps of Engineers to be 5,000-7,000 cfs and 1,500-2,500 cfs,respectively (see plate A-2 for location).

IV. SYNTHESIS OF STANDARD PROJECT FLOOD

4.01. GENERAL. The standard project flood (SPF) represents the flood that would result from the most severe combination of meteorologic and hydrologic conditions considered reasonably characteristic of the region. It normally is larger than any past recorded flood in the area, and can be expected to be exceeded in magnitude only on rare occasions.

4.02. STANDARD PROJECT STORM. The thunderstorm that occurred at Indio on 24 September 1939 is considered to be the most severe local storm that could reasonably be expected to occur in the area. This storm was therefore used to determine the standard project flood for the basin. The average precipitation for the basin during the storm was determined by reducing the point precipitation (6.45 inches) for the Indio storm to average precipitation over the basin by means of the depth-area curve (see plate A-4) developed from the isohyetal analysis shown on plate A-5, which happens to be almost exactly parallel to the depth-area curve developed for 3-4 March 1943 storm that occurred in the Los Angeles area. The precipitation-intensity pattern for the local storm was determined from a mass curve of observed rainfall during the 24 September 1939 storm at Indio (see plate A-6). A typical precipitation-intensity pattern is shown on plate A-11.

4.03. RAINFALL-RUNOFF RELATIONSHIPS.

a. There are no precipitation and runoff records available for an analysis of rainfall-runoff relationships in the Magnesia Spring Canyon basin. The rainfall-runoff relationships adopted for this study were taken

from reference 1. Elements used to establish rainfall-runoff relationships are discussed in the following paragraphs.

b. Unit Hydrographs. The unit hydrograph procedure used by the Los Angeles District has its basis in an S-graph, which is the time distribution of runoff as a function of basin lag time. Lag time is defined as the time in hours for 50 percent of the total volume of runoff of the unit hydrograph to occur. The basin lag time can be approximated for ungaged watersheds by the use of the lag relationship presented on plate A-7. The basin n value is a proportionality factor in the equation for lag time which permits adjustment of lag time depending on type of ground cover and surface characteristics affecting basin response to effective rainfall. Synthetic unit hydrographs were determined from the S-graph shown on plate A-8. Pertinent characteristics for subareas used in this study are presented in table A-3.

c. Baseflow and Rainfall Loss Rate. Baseflow was considered negligible during the standard project storm. A constant loss rate of 0.20 inch per hour was adopted in this study, with a factor to account for impervious areas, such as roads and rock outcrops.

4.04. FLOOD ROUTING.

a. Because the upstream levees would fail under the large floods of interest, no routing was done for the existing West Magnesia Spring channel. Overflow boundaries through Rancho Mirage will be developed from observed data obtained from the July 1979 flood.

b. Flood routing in improved channels was accomplished by the Muskingum method of channel routing. Flood wave travel time in a reach, which

approximates the Muskingum coefficient K, was determined by dividing reach length by average peak flow velocity. Manning's formula for normal flow and a preliminary design cross-section were used to compute the average peak flow velocity for the reach. An X value of 0.4 was used for the proposed concrete channel. Muskingum coefficients are given in table A-3.

4.05. COMPUTATION OF STANDARD PROJECT FLOOD.

a. Standard project flood was computed by centering the standard project storm in the most critical flood producing manner. Application of the constant loss rate to the standard project precipitation enables determination of the rainfall excess. The rainfall excess is then applied to the subbasin unit hydrograph to produce the subbasin flood hydrograph. Combining and routing of subbasin flood hydrographs to the desired concentration point completes the computation of a standard project flood.

b. Standard project flood peak discharges, computed as described above, are presented in tables A-1 and A-2 and shown on plates A-9 and A-10 for pre-project and project conditions, respectively. The standard project flood hydrograph at the debris basin site is shown on plate A-11.

V. SYNTHESIS OF PROBABLE MAXIMUM FLOOD

5.01. GENERAL. The probable maximum flood (PMF) is defined as the flood that would result if the probable maximum precipitation for the drainage area were to occur at a time when ground conditions were conducive to maximum runoff. Probable maximum flood, as its name implies, is an estimate of the upper bound of flood potential on a watershed. Such a hypothetical flood is necessary for proper design of debris basin and dam spillways.

5.02. PROBABLE MAXIMUM PRECIPITATION. Probable maximum precipitation (PMP) is considered the practical upper limit of available precipitable water over an area as estimated by the Hydrometeorological Branch of the National Weather Service. Local storm PMP estimates were computed from Hydrometerological Report (HMR) NO. 49, "Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages," dated September 1977. Computation of PMP is shown on plate A-12.

5.03. PROBABLE MAXIMUM FLOOD. Computation of PMF was accomplished in the same manner as SPF, with two exceptions. First, basin lag time was reduced by 15 percent to account for the reduction in the response time of rainfall excess characteristic of large floods where the hydraulic efficiency of the watershed is increased by high depths of flow. Secondly, the loss rate was reduced to 0.15 inch per hour. This is a minimum loss rate deemed reasonable of a watershed saturated by antecedent rainfall. The PMF peak discharge and volume at the debris basin site (flood control dam site) are 44,000 cfs and 4,390 ac-ft, respectively; the hydrograph is shown on plate A-12.

VI. DEBRIS PRODUCTION

6.01. The location of the proposed debris basin is at the mouth of the Magnesia Spring Canyon, approximately 1.3 miles upstream the Whitewater River confluence and near elevation 480 feet (see plate A-2). A quantitative estimate of the debris production from a single, large storm event was computed using the Tatum method (reference 2). Measurements of slope, drainage density, and hypsometric index were obtained from available 1:24,000 topographic maps. Corps of Engineers geologists have determined that the overall debris potential of the basin is low. In light of the low debris potential and the lack of significant ground vegetation, the best estimate of debris production would be obtained by applying the correction factors to the ultimate debris production value estimated for 10 years after a burn. The estimated total debris production from a single, large storm event is 150,000 cubic yards. Debris production parameters used in the analysis are given in table A-4.

VII. DISCHARGE FREQUENCY ANALYSIS

7.01. GENERAL. Discharge frequency analysis in the study area involved determination of discharge frequency values with and without the proposed improvements. No streamgages exist in the study watershed; thus, regional discharge frequency relationships developed in reference 1 were used in the analysis. Estimates from rainfall-runoff calculations were also included.

7.02. REGIONAL FREQUENCY ANALYSIS. N-year peak discharges from the individual station frequency curves for 8 stream gages within the Whitewater River Basin (developed for ref. 1), stated in cfs per square mile, were plotted versus drainage area. N-year peak discharges computed from rainfall for some of the streams were also plotted. A smooth family of curves representing peak discharge per square mile for the 500-, 100-, 50-, and 10-year return periods, and for the standard project flood, were then drawn through the plotted points. The results are reproduced on plate A-14. Stream gage station data and peak discharge statistical parameters are also reproduced in tables A-6 and A-7, respectively. A more detailed discussion of the regional frequency analysis can be found in reference 1.

7.03. ESTIMATES FROM RAINFALL. A 100-year flood peak discharge was computed for the subarea above the debris basin site using a runoff model and a hypothetical storm. The hypothetical 100-year storm was based on the standard project storm pattern, with t-hour amounts adjusted so as not to exceed the 100-year rainfall statistics determined by the National Weather Service (reference 3). The computed 100-year peak discharge of 4,300 cfs compares

favorably with the adopted value, which was based on the regional frequency curves.

7.04. ADOPTED RELATIONSHIPS.

a. Peak discharge frequency values were determined from the regional frequency curves. These values were then adjusted to reflect the slightly lower runoff potential of the basin indicated by a comparison of the computed SPF and SPF estimated from the regional curves. The adopted discharge frequency values for pre-project conditions are given in table A-1. The discharge frequency curve at the debris basin site is shown on plate A-15.

b. Peak discharges for project conditions were determined by routing and combining n-year flood subarea hydrographs, reduced by the ratio of n-year peak to SPF peak, as determined from the adopted frequency curve shown on plate A-15. N-year peak discharge values for project conditions are given in table A-2.

c. N-year peak discharges for the small tributaries in the project area are also given in table A-2. These values were computed as described above and are considered suitable for side drainage design.

BIBLIOGRAPHY

1. Hydrology for Feasibility Report, Whitewater River Basin, Corps of Engineers, Los Angeles District, May 1980.

2. New Method of Estimating Debris-Storage Requirements for Debris Basin, by Fred E. Tatum, U.S. Army Corps of Engineers, Los Angeles District, January 1963.

3. NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume XI-California, National Oceanic and Atmospheric Administration, 1973.

4. Draft Engineering Report on Preliminary Design and Cost Estimate for Flood Control Works for Palm Desert--Rancho Mirage--Indian Wells, Apendix A, Hydrology, Bechtel Inc., for Coachella Valley County Water District, June 1977.

5. Flood Damage Report, San Bernardino, Riverside, Imperial Counties, California, Floods of September 1976, U.S. Army Corps of Engineers, Los Angeles District, September 1977.

TABLE A-1

PEAK DISCHARGES

WITHOUT PROJECT

Concentration Point	Drainage Area (mi^2)	Peak Discharge (cfs) SPF	500-Yr	100-Yr	50-Yr	10-Yr
Magnesia Spring Canyon:						
CP 1-A* (Whitewater River)	-	-	-	-	-	-
CP 4 (Elev. 480 ft.)	4.9	6,600	12,000	4,200	2,700	570
Whitewater River:						
at Rancho Mirage	720	78,000	90,000	37,000	22,000	6,100

* Hydraulics Section will develop the overflow boundaries from discharge-depth relationships determined from observed depths during the July 1979 flood and the July 1979 estimated discharges at the canyon mouth (CP 4). Therefore, discharge frequency estimates of CP 4 given above are sufficient for overflow determinations.

TABLE A-2

PEAK DISCHARGES

WITH PROJECT

Concentration Point	Drainage Area (mi^2)	Peak Discharge (cfs)				
		SPF	500-Yr	100-Yr	50-Yr	10-Yr
Magnesia Spring Canyon:						
CP 1 (Mouth)	5.3	6,800	6,800*	4,300	2,600	590
CP 2 (Below confl. with Stream "A")	5.3	6,800	6,800*	4,300	2,600	590
CP 3 (Below confl. with Stream "B")	5.1	6,600	6,600*	4,200	2,500	570
CP 4 (Elev. 480 ft.)	4.9	6,600	12,000	4,200	2,500	570
Stream "A" (at Magnesia Spring)	0.25	500	1,000	350	200	40
Stream "B" (at Magnesia Spring)	0.15	300	600	200	120	25

* Flows greater than design discharge (SPF) proceed downslope through Rancho Mirage.

TABLE A-3

SUBAREA CHARACTERISTICS

Subarea	Drainage Area (mi^2)	L (mi)	Lca (mi)	Slope (ft/mi)	n	% Impervious
A1	0.25	1.02	0.76	765	0.035	5
A2	0.15	0.76	0.34	895	0.035	5
B	4.90	3.13	1.40	600	0.035	5

ROUTING COEFFICIENTS

Routing Reach	T (hrs)	Muskingum Coefficients NRCHS	K (hrs)	X
CP 4-CP 1	0.083	1	0.083	0.4

T = Routing time interval

NRCHS = Number of subreaches

TABLE A-4

MAGNESIA SPRING CANYON

DEBRIS PRODUCTION PARAMETERS[4]

	Drainage Area (mi^2)	Slope (ft/mi)	Drainage Density[1] (mi)	Hypsometric Index[2]	3-Hour Rainfall (in)
	4.9	600	1.76	0.51	3.5
Correction Factor[3] (%)	-	67	97	99	100

Ultimate debris production = 48,000 cu. yd./sq. mi.

Total correction factor = 64%

Total debris production = 150,000 cubic yards

(1) Total streams length in miles, divided by the drainage area in square miles.

(2) Relative height at which the drainage area is divided into two equal parts.

(3) Percentage for each of the parameters that represents the difference between ultimate and actual conditions.

(4) Tatum method.

TABLE A-5

OBSERVED PRECIPITATION AT INDIO

STORM OF 24 SEPTEMBER 1939

Time Hours	Accumulated Precipitation Inches
0500	0
0800	2.00
0930	3.70
1015	5.45
1100	6.45

TABLE A-6
STREAMGAGING STATIONS IN THE WHITEWATER RIVER BASIN

U.S.G.S. Gage No.	Location	Drainage Area sq. mi.	$H^{(3)}$	Period of Record	$N^{(4)}$	Maximum Peak Discharge Date	Maximum Peak Discharge cfs
10256000	Whitewater River at Whitewater, California	57.5	62	1950-1977	28	11-22-65 3-02-38 2- -27	24,000 42,000* 6,000*
10256400	San Gorgonio River near Whitewater, California	154		1966-1978	13	1-25-69	7,250
10256500	Snow Creek near Whitewater, California	10.8	63	1961-1978	18	1-25-69 2- -27	13,000 9,500*
10257600	Mission Creek near Desert Hot Springs, California	35.7		1968-1977	10	1-25-69	1,660
10258000	Tahquitz Creek near Palm Springs, California	16.8		1948-1978	31	1-25-69 11-22-65	2,900 2,900
10258500	Palm Canyon Creek near Palm Springs, California	93.3		1930-1941 1948-1978	43	9-10-76 2- -27	4,050 9,400*
10259000	Andreas Creek near Palm Springs, California	8.6		1950-1978	29	8-31-54	1,960
10259200	Deep Creek near Palm Desert, California	30.6		1963-1978	16	9-10-76	7,100

TABLE A-6 (Continued)

U.S.G.S. Gage No.	Location	Drainage Area sq. mi.	H(3)	Period of Record	N(4)	Maximum Peak Discharge Date	Maximum Peak Discharge cfs
10259300	Whitewater River at Indio, California	1,073	63	1966-1978	13	11-22-65	14,100
						3-03-38	29,000*
						2- -27	10,000*
						1- -16	10,000*
10259540	Whitewater River near Mecca, California	1,299		1961-1976	5	1-25-69	2,500(1)
						9-10-78	6,200(2)
10257800	Long Creek near Desert Hot Springs, California	19.4		1963-1976	14	8-07-63	9,270

(1) Maximum daily flow. Only daily flows available since 1966.

(2) Estimated peak (ref. 5).

(3) Historic record length.

(4) Systematic record length.

* Estimated.

TABLE A-7
PEAK DISCHARGE STATISTICAL PARAMETERS

U.S.G.S Gage No.	Location	Drainage Area	Based on Adjusted Records				Based on Systematic Records		
			H	X	S	N	X	S	G
10256000	Whitewater River at White-water, California	57.5 (1)	62	2.6658	0.6301	28	2.6808	0.6531	1.04
10256400	San Gorgonio River near Whitewater, California	154				13	2.5434	0.8033	-0.07
10256500	Snow Creek near Whitewater, California	10.8 (2)	63	2.4993	0.6375	18	2.5411	0.6972	0.95
10257600	Mission Creek, near Desert Hot Springs, California	35.7 (3)				10	1.2023	1.2639	0.50
10257800	Long Creek near Desert Hot Springs, California	19.4 (3)				14	1.6959	1.2239	0.65
10258000	Tahquitz Creek near Palm Springs, California	16.8 (4)	31	2.0769	0.6942	31	1.8751	0.7893	0.45
10258500	Palm Canyon Creek near Palm Springs, California	93.3 (5)	43	2.3295	0.8626	43	2.3541	0.8546	-1.06
10259000	Andreas Creek near Palm Springs, California	8.6 (6)		1.8718	0.76	29	1.8718	0.5816	1.03

TABLE A-7 (Continued)

U.S.G.S Gage No.	Location	Drainage Area	Based on Adjusted Records			Based on Systematic Records			
			H	X	S	N	X	S	G
10259200	Deep Creek near Palm Desert, California	30.6(7)	16	2.2955	0.7894	16	2.0203	1.2566	-0.97
10259300	Whitewater River at Indio, California	1073(8)				13	2.6590	1.2145	-0.09

(1) Historical period = 1916-77; historical flows = 1927 and 1938.

(2) Historical period = 1916-78; historical flow = 1927.

(3) Contains many zeros in systematic records; statistics are for non-zero flow record.

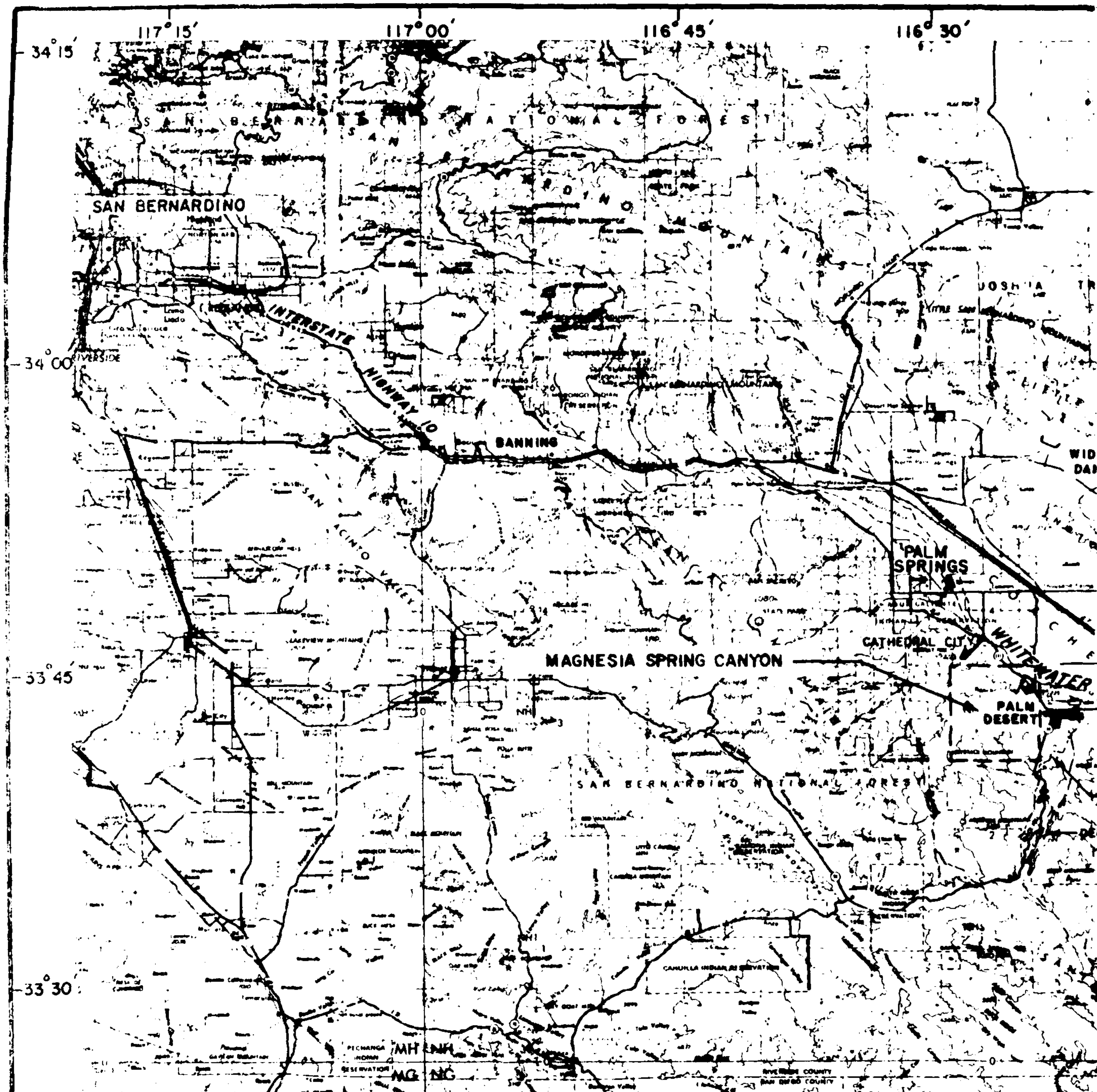
(4) Adjusted statistics are statistics for incomplete record; five flows less than or equal to 10 cfs truncated.

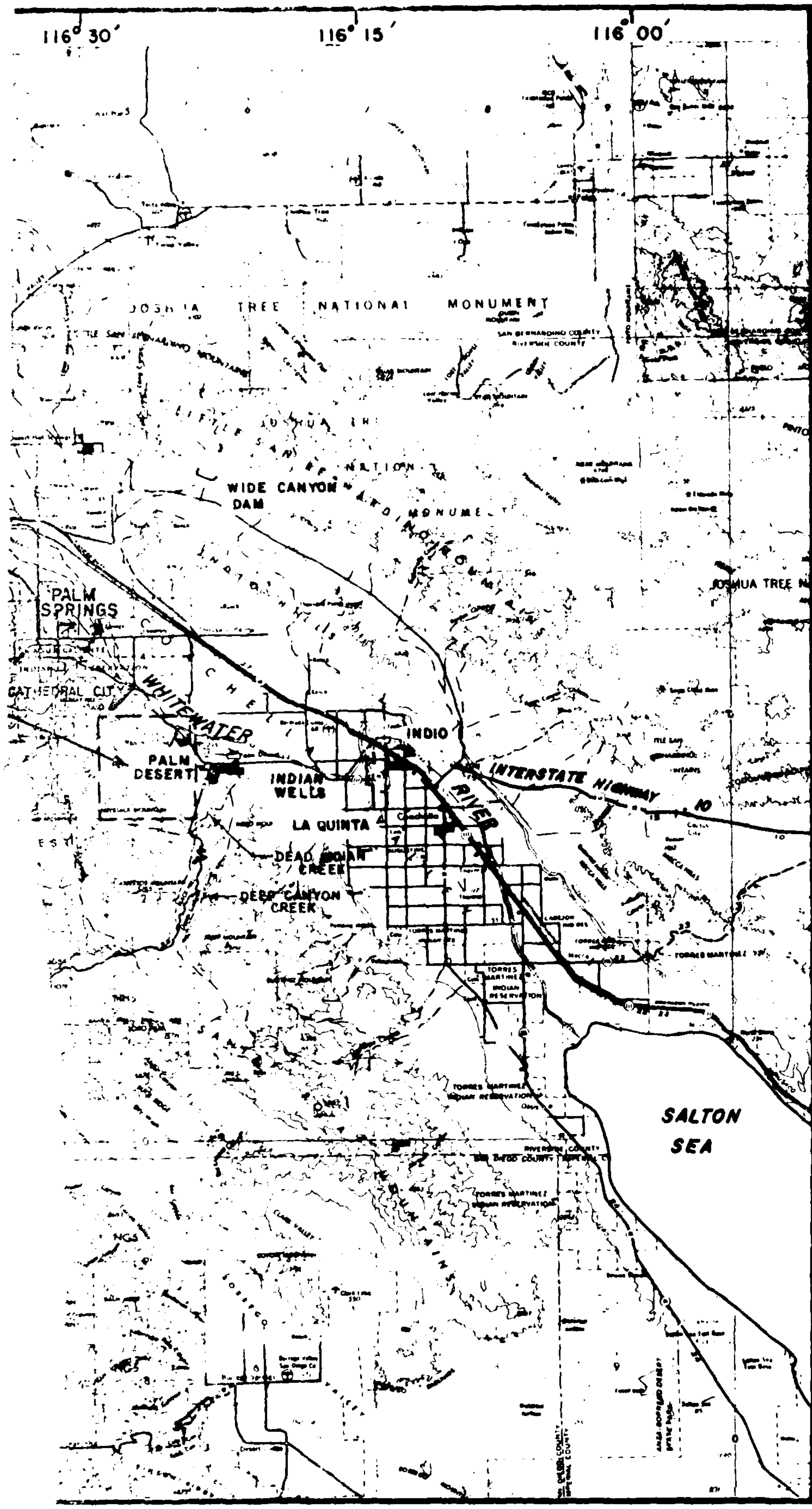
(5) Synthetic statistics for zero flow adjustment; one zero flow. Record is broken. Statistics for systematic record are for non-zero flow record.

(6) Refer to discussion in para. 7-02f, ref. 1.

(7) Synthetic statistics for incomplete record adjustment; three flows less than or equal to 10 cfs truncated.

(8) Refer to discussion in para. 7-02g, ref. 1. Contains two zeros in systematic record; statistics are for non-zero flow record.


117° 15′
117° 00′
116° 45′
116° 30′
34° 15′
34° 00′
33° 45′
33° 30′
33° 15′
SAN BERNARDINO
RIVERSIDE
INTERSTATE HIGHWAY 10
BANNING
SAN JACINTO VALLEY
PALM SPRINGS
CATHEDRAL CITY
WHITEWATER
MAGNESIA SPRING CANYON
PALM DESERT
SAN BERNARDINO NATIONAL FOREST
SAN BERNARDINO MOUNTAINS
CLEVELAND NATIONAL FOREST
MH NH
MG NG



N

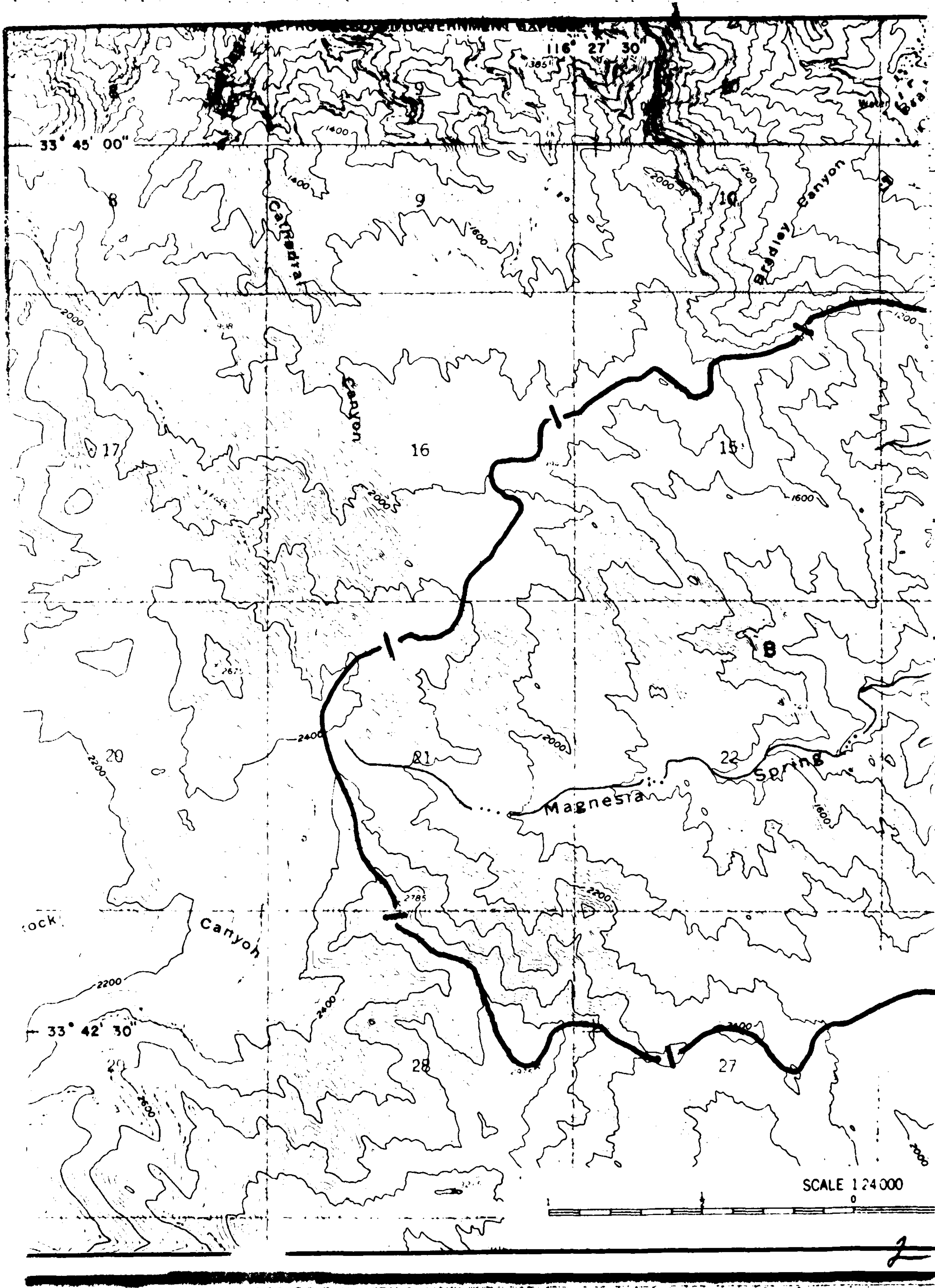
SCALE: 0 5 10 15 MILE

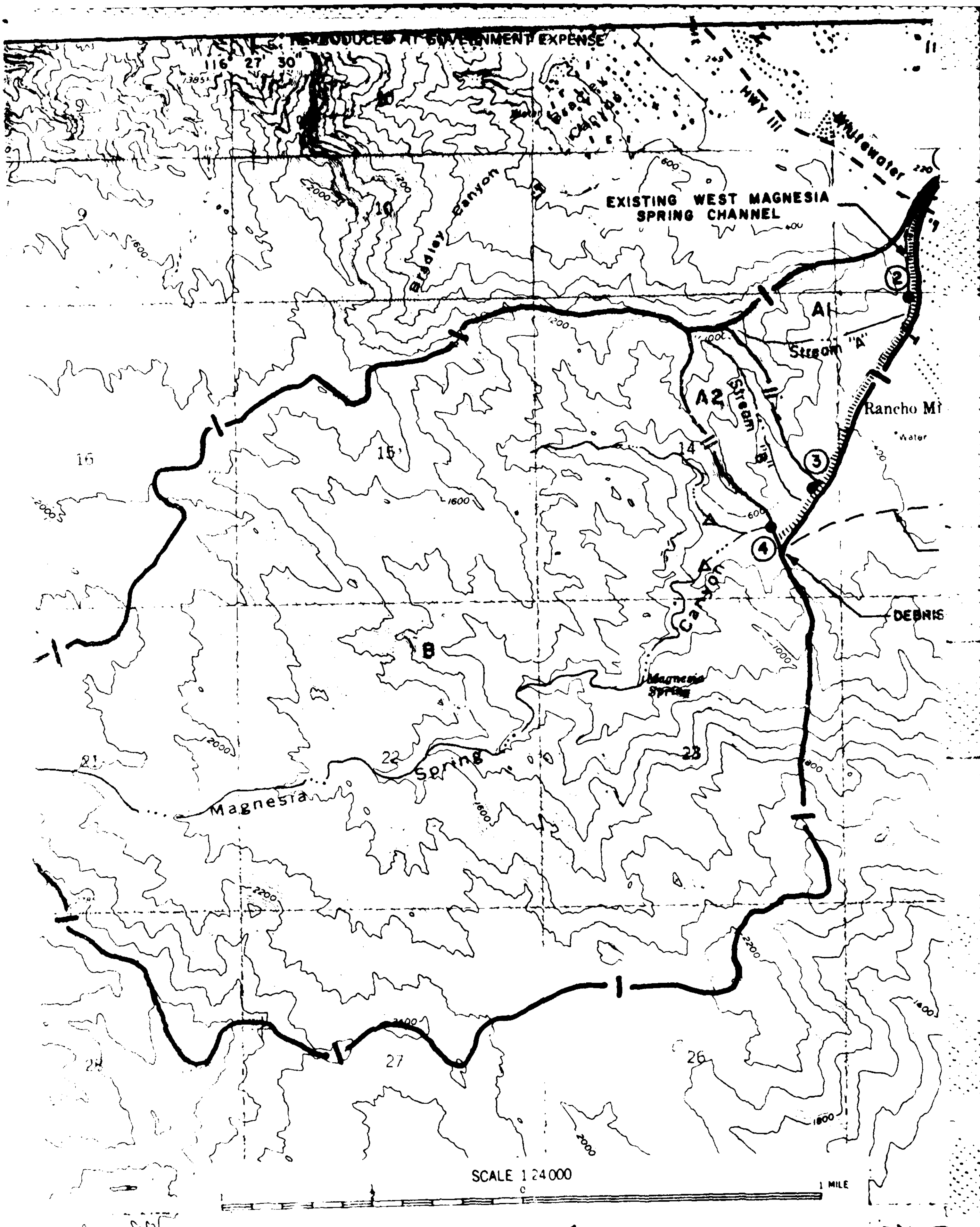
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

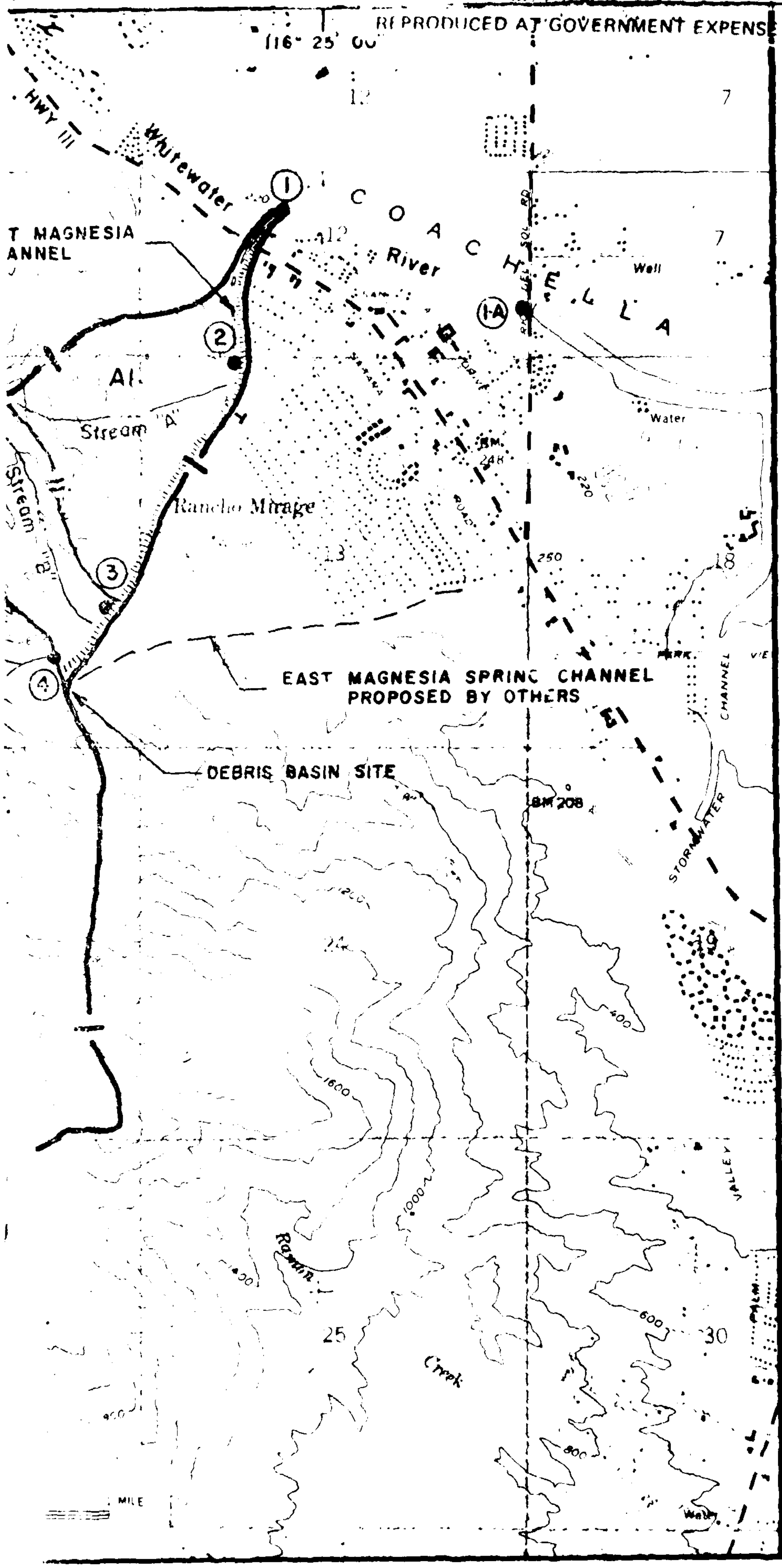
GENERAL LOCATION

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE II


116° 27' 30"
33° 45' 00"
8
9
10
Cathedral
Canyon
Bradley Canyon
17
16
15
B
20
21
22
Magnesia
Spring
Canyon
33° 42' 30"
29
28
27
SCALE 1:24 000
2000
1600
1400
1200
2200
2400
2600
2785
2675


REPRODUCED AT GOVERNMENT EXPENSE
116° 27' 30"
Bradley Canyon
HWY 111
Whitewater
EXISTING WEST MAGNESIA
SPRING CHANNEL
A1
Stream "A"
A2
Stream "B"
Rancho Mi
Water
DEBRIS
Canyon
Magnesia Spring
B
Magnesia Spring
SCALE 1:24000
1 MILE



LEGEND

BOUNDARY OF DRAINAGE AREA

BOUNDARY OF DRAINAGE SUBAREA

B SUBAREA DESIGNATION

CONCENTRATION POINT AND/OR LOCATION NUMBER

EXISTING LEVEE

LOCATION OF JULY 1979 FLOOD ESTIMATES

MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

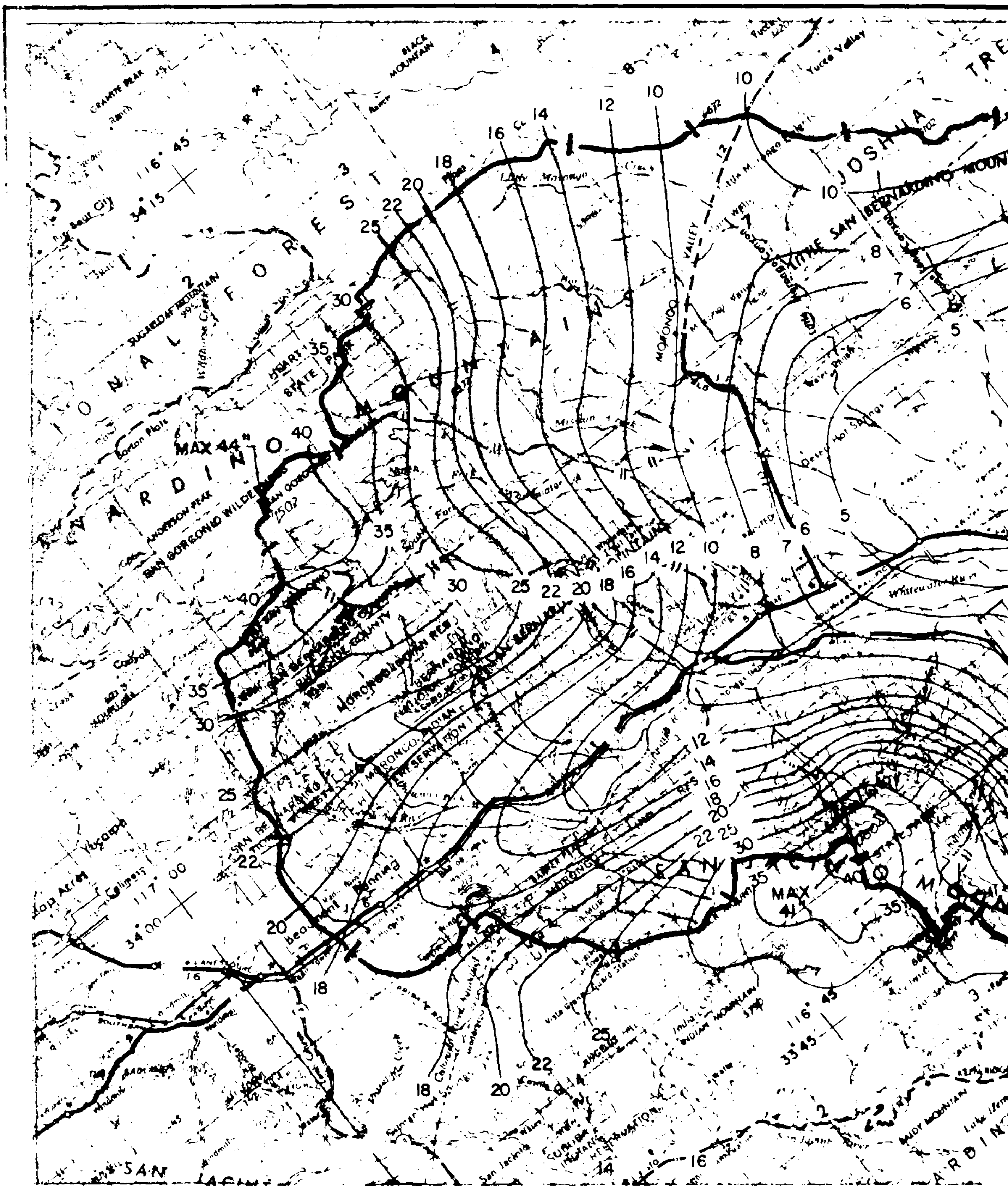
DRAINAGE AREA BOUNDARIES

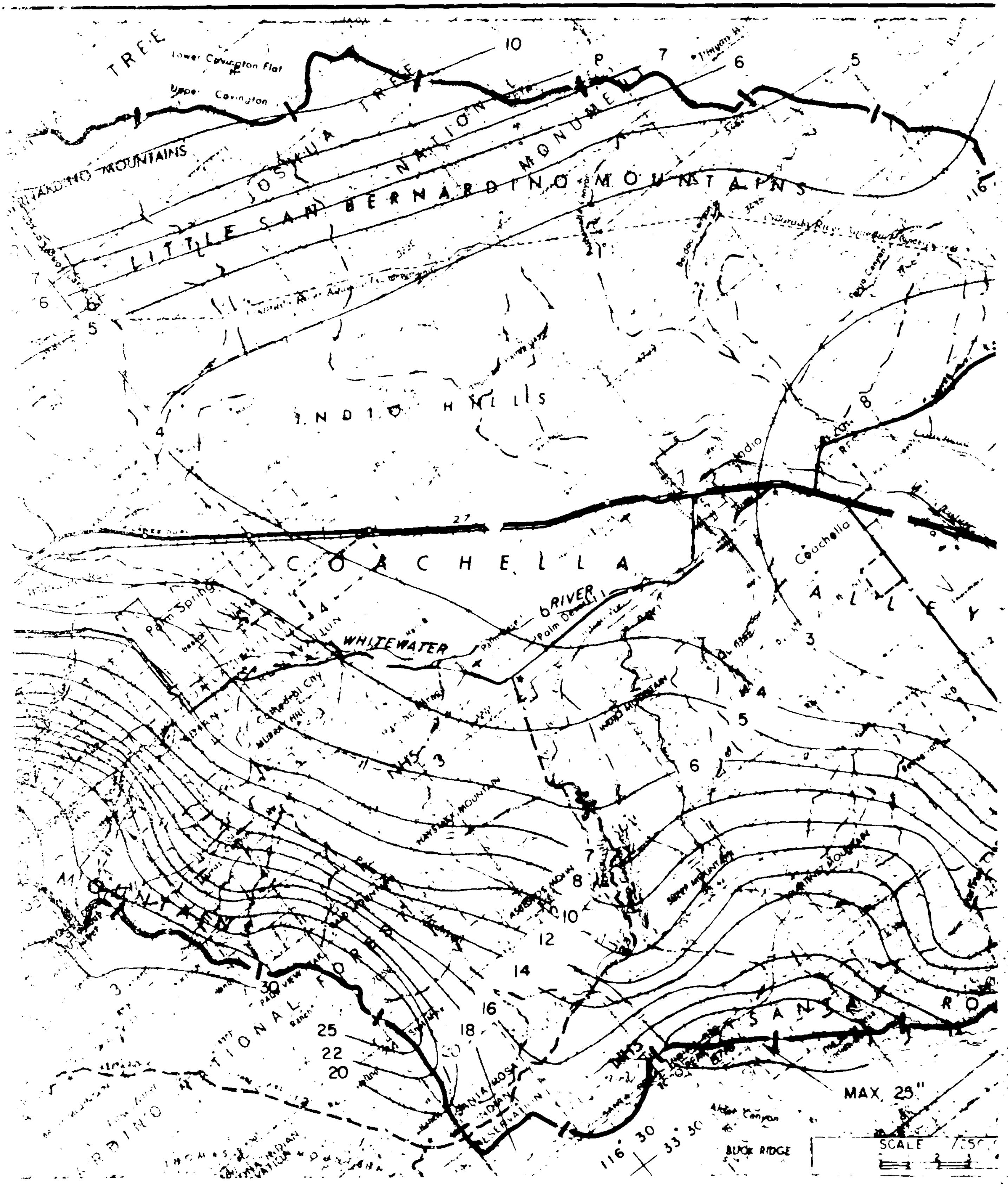
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

II

PLATE

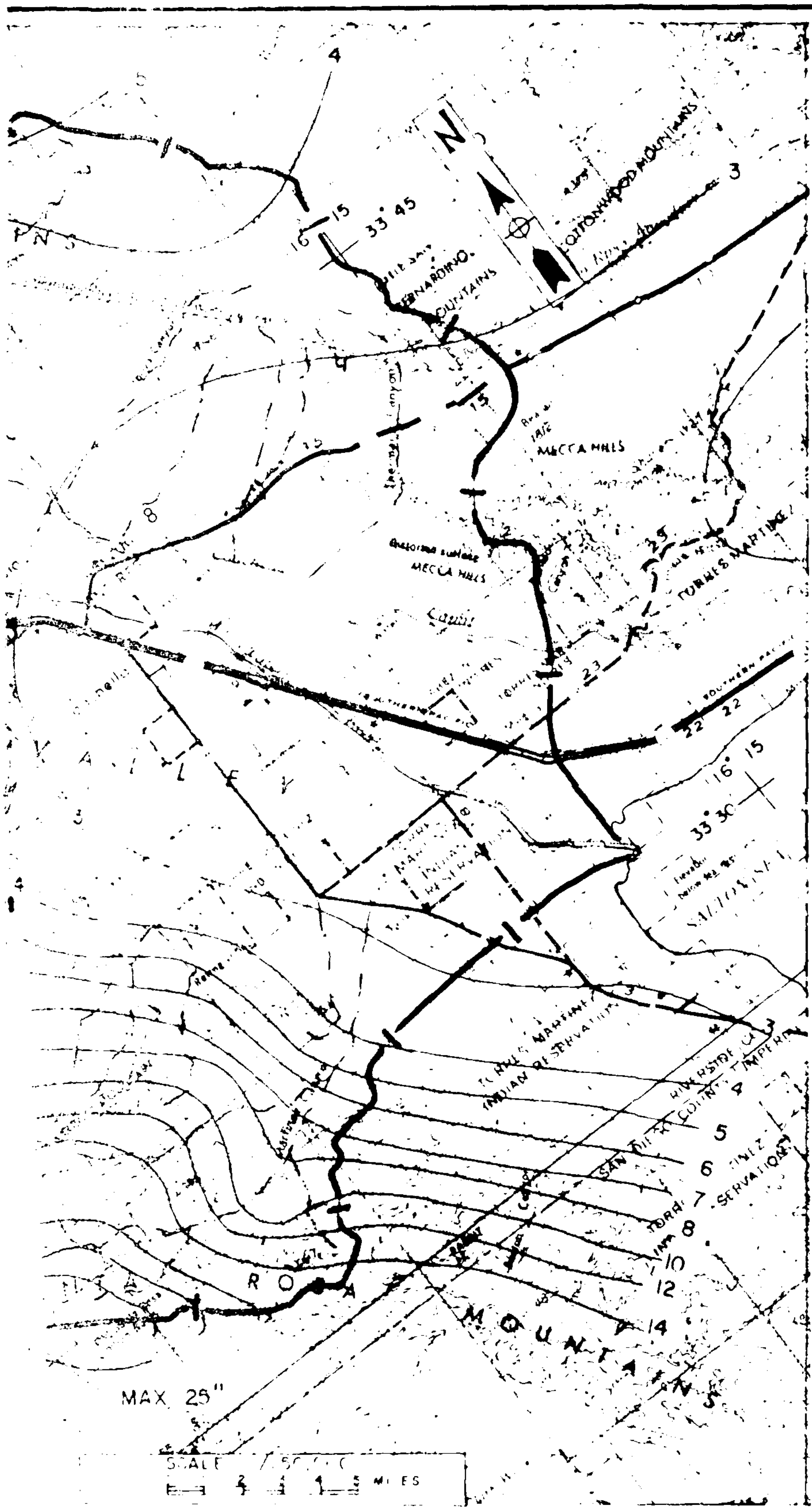
REPRODUCED AT GOVERNMENT EXPENSE





Lower Covington Flat
Upper Covington
JOSHUA TREE NATIONAL MONUMENT
LITTLE SAN BERNARDINO MOUNTAINS
INDIO HILLS
Indio
COACHELLA VALLEY
Coachella
Palm Springs
WHITEWATER RIVER
Cathedral City
Palm Desert
SANTA ROSA INDIAN RESERVATION
Alder Canyon
BUCK RIDGE
MAX. 25"
SCALE
116° 30'
33° 30'

II



LEGEND

BOUNDARY FOR WHITEWATER RIVER BASIN

— 12 — ISOHYETAL LINE IN INCHES

NOTE:

MEAN ANNUAL ISOHYETS BASED ON COMBINED DATA OF 1878-79 SEASON TO 1953-54 (FROM 1961 U.S. A.C.E. REPORT) AND 1935-60 (FROM 1973-74 RIVERSIDE CO. F.C.D. REPORT)

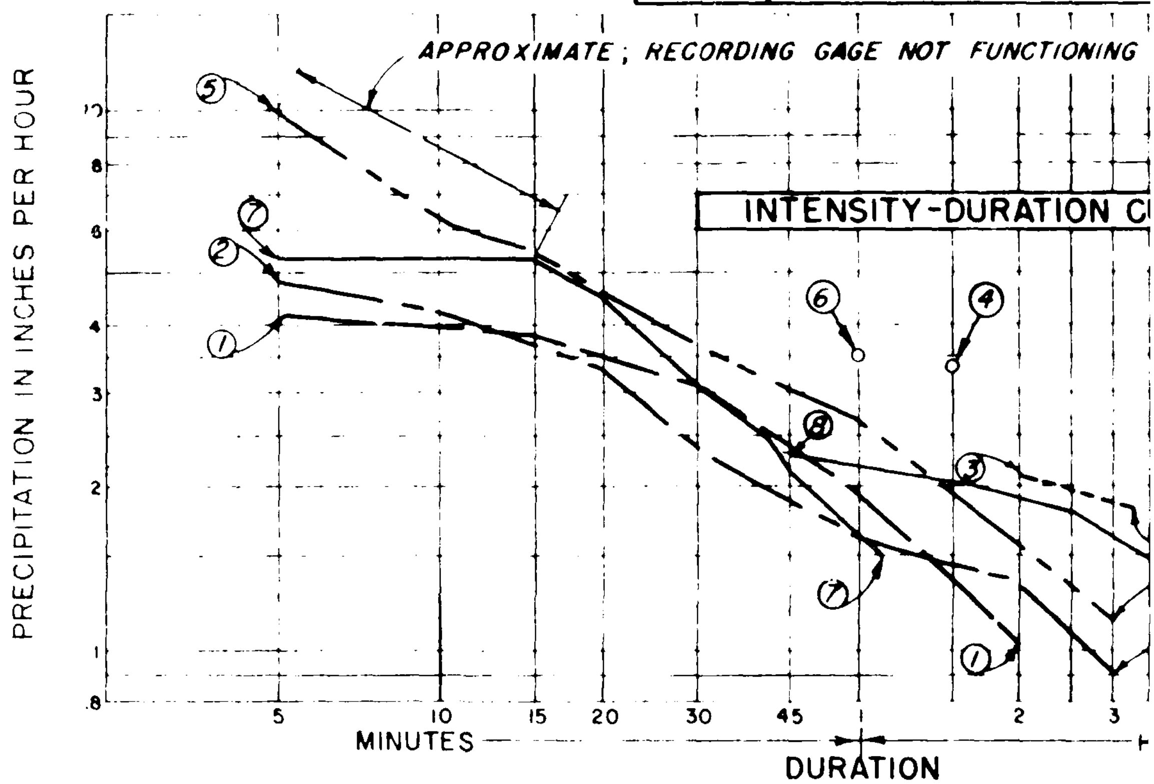
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

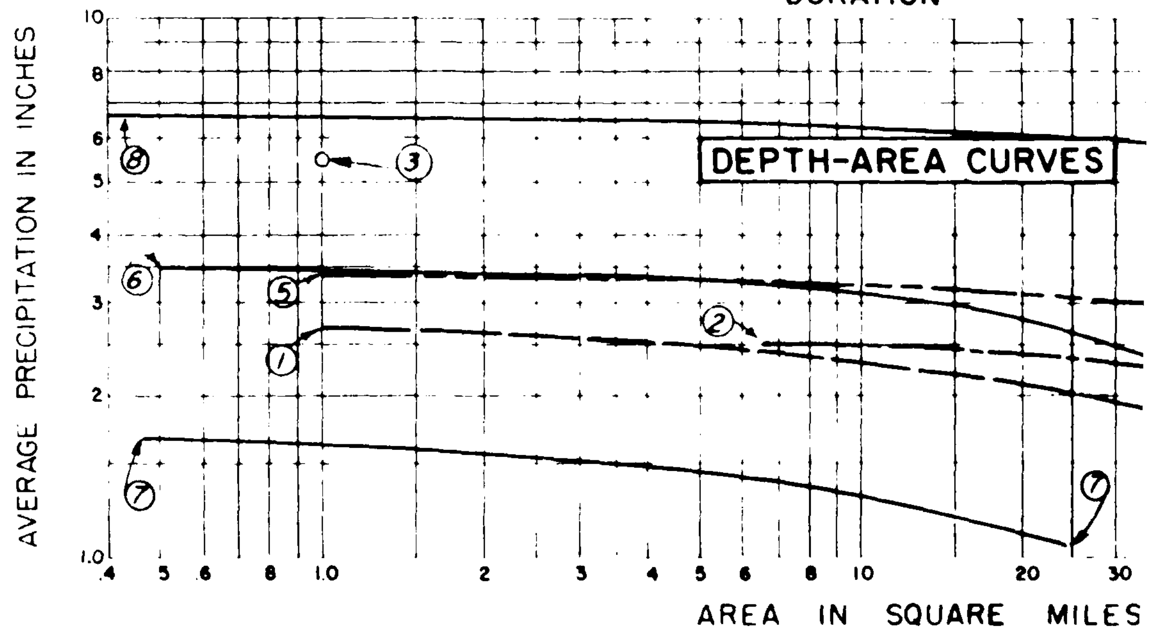
MEAN ANNUAL ISOHYETS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

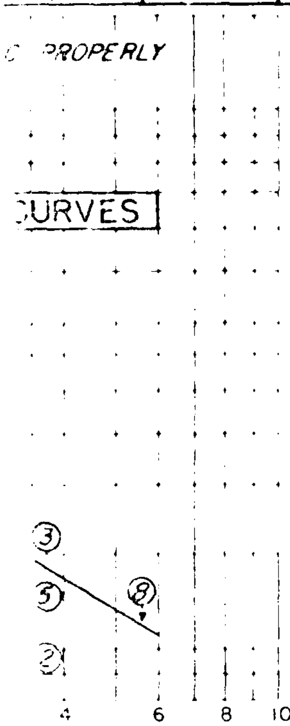
PLATE A-3

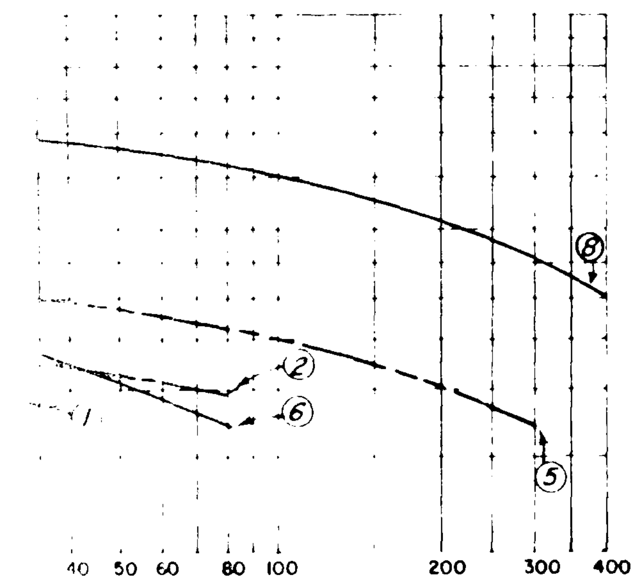
CURVE NUMBER	PRECIPITATION S… NAME
1	SUNNY HILLS RANCH
2	TOPANGA CANYON RANGER ST
3	AVALON
4	SQUIRREL INN
5	SIERRA MADRE – CARTER
6	GARRET WINERY
7	SANTA BARBARA (FIRE STA
8	INDIO (REVISED 7 MAR 7

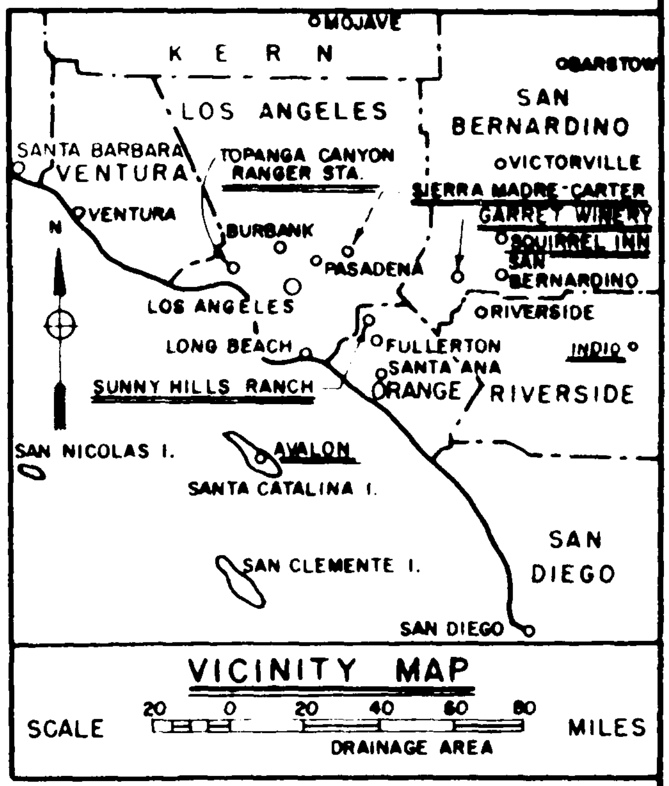




STATION		STORM			
	NUMBER	LOCATION	DURATION		DATE
			HR	MIN.	
	8P98	FULLERTON, CALIF.	2	0	MAR. 14, 1941
STATION	7077	TOPANGA CANYON, CALIF.	3	0	FEB. 20, 1941
	7P10	AVALON, CALIF.	3	15	OCT. 21, 1941
	8012	SQUIRREL INN, CALIF.	1	30	JULY 18, 1922
	701338	SIERRA MADRE, CALIF.	3	0	MAR. 3-4, 1943
	—	CUCAMONGA, CALIF.	1	0	SEPT. 29, 1946
A #3)	—	SANTA BARBARA, CALIF.	1	10	FEB 4, 1958
3)	9P13	INDIO, CALIF.	6	0	SEPT. 24, 1939







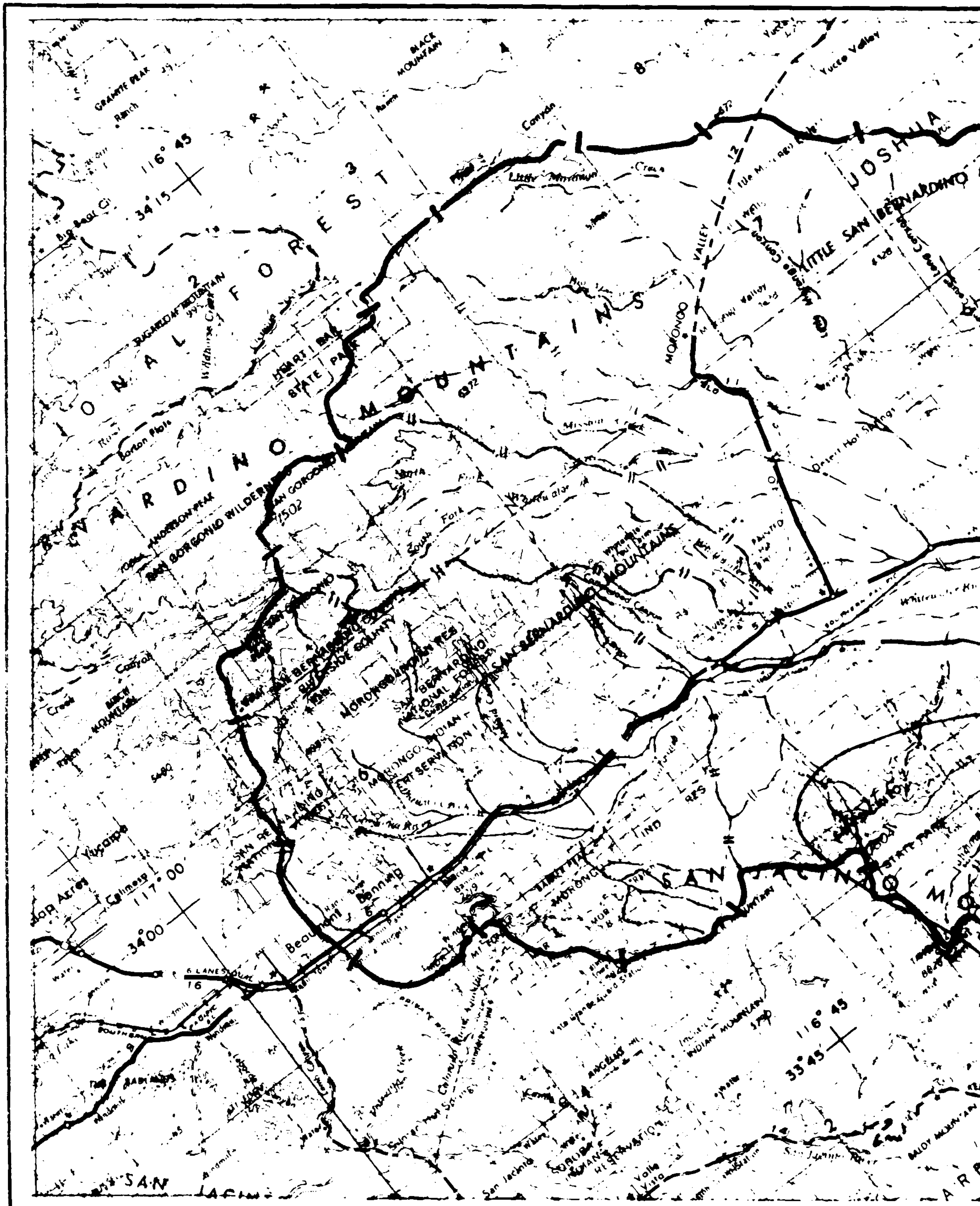
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

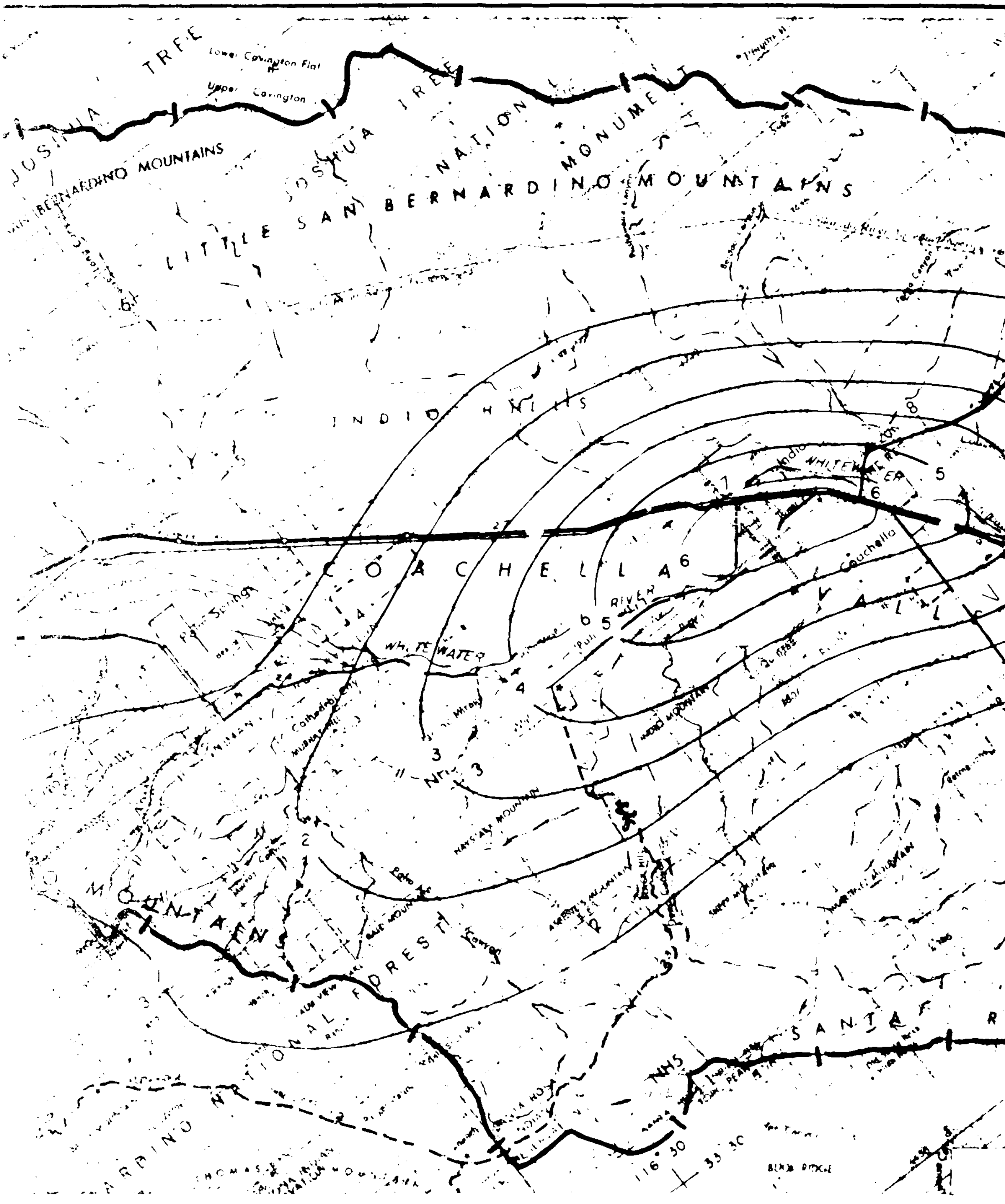
INTENSITY-DURATION AND DEPTH-AREA RELATIONSHIPS LOCAL STORMS IN SOUTHERN CALIFORNIA

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED:

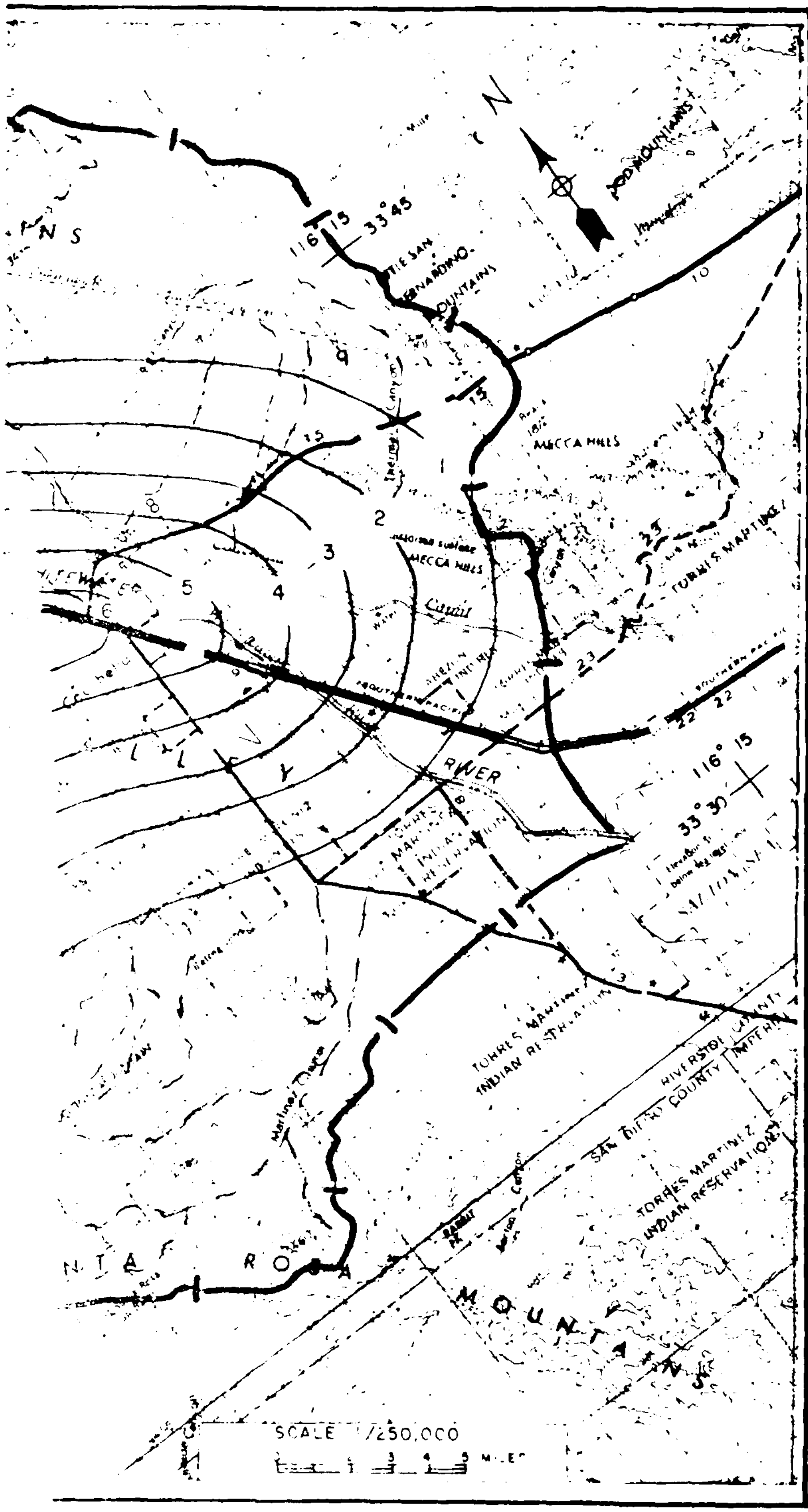
PLATE A-4

II


GRANITE PEAK
BLACK MOUNTAIN
116° 45
34° 15
Big Bear City
NATIONAL FOREST
SAN BERNARDINO MOUNTAINS
SUGARLOAF MOUNTAIN
Barton Flats
HEART BAR
STATE PARK
ANDERSON PEAK
SAN GORGONIO WILDERNESS
SAN GORGONIO
11502
JOSHUA
LITTLE SAN BERNARDINO
MORONGO VALLEY
Yucca Valley
Desert Hot Springs
Little Morongo Creek
Mission Creek
South Fork
Whitewater
SAN BERNARDINO COUNTY
RIVERSIDE COUNTY
MORONGO INDIAN RES
SAN BERNARDINO NATIONAL FOREST
Yucaipa
Calimesa
117° 00
34° 00
Beaumont
Banning
6 LANES
16
SAN JACINTO MOUNTAINS
STATE PARK
INDIAN MOUNTAIN
116° 45
33° 45
INDIAN RESERVATION
San Jacinto
Valle Vista
SAN JACINTO


JOSHUA TREE
Lower Covington Flat
Upper Covington
JOSHUA TREE NATIONAL MONUMENT
SAN BERNARDINO MOUNTAINS
LITTLE SAN BERNARDINO MOUNTAINS
INDIO HILLS
COACHELLA VALLEY
WHITEWATER RIVER
WHITEWATER
Palm Springs
Indio
Coachella
INDIO MOUNTAIN
SANTA ROSA MOUNTAINS
SAN BERNARDINO NATIONAL FOREST
116° 30
33 30
BLACK RIDGE

II



LEGEND

BOUNDARY FOR WHITEWATER RIVER BASIN

3 — ISOHYETAL LINE IN INCHES

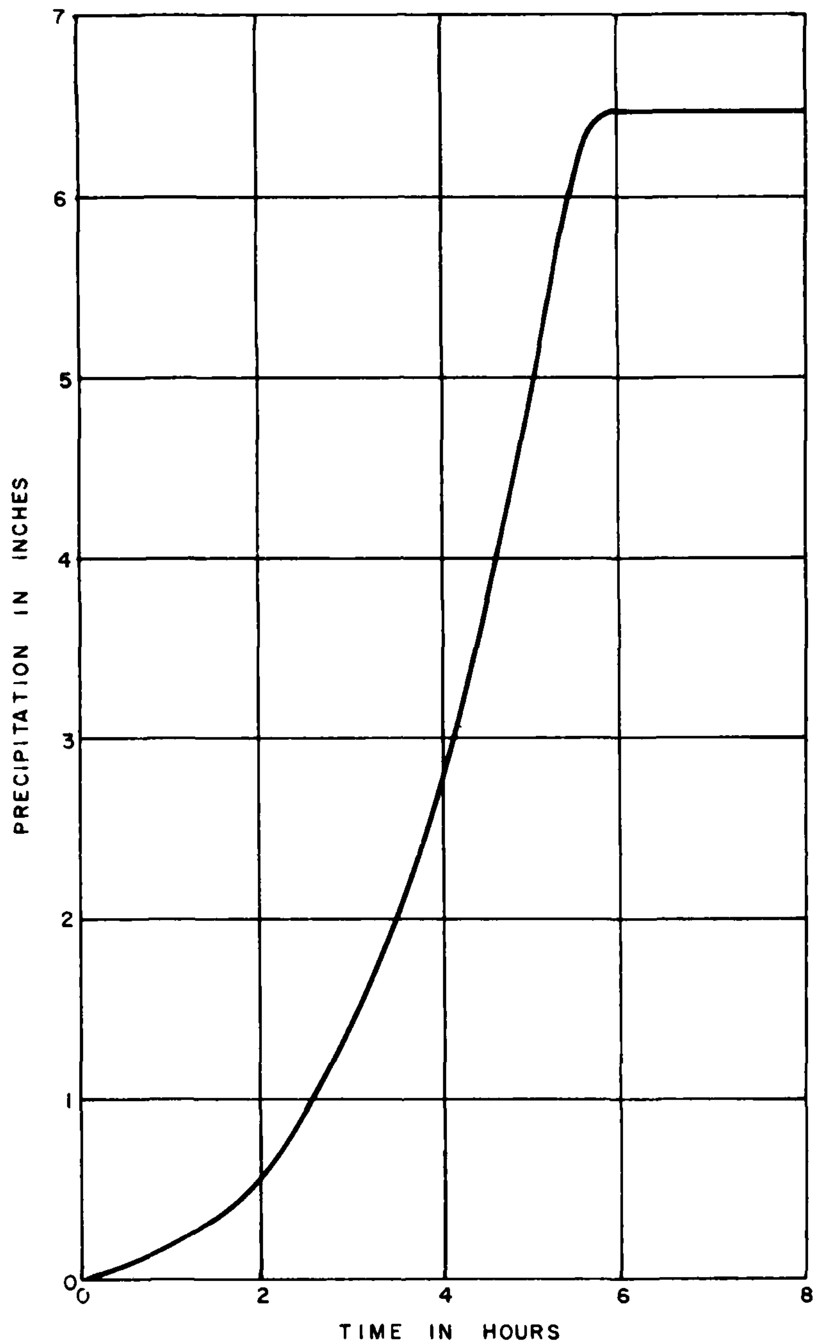
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

TOTAL STORM ISOHYETS
STORM OF 24 SEPTEMBER 1939

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED:

PLATE A-5

III

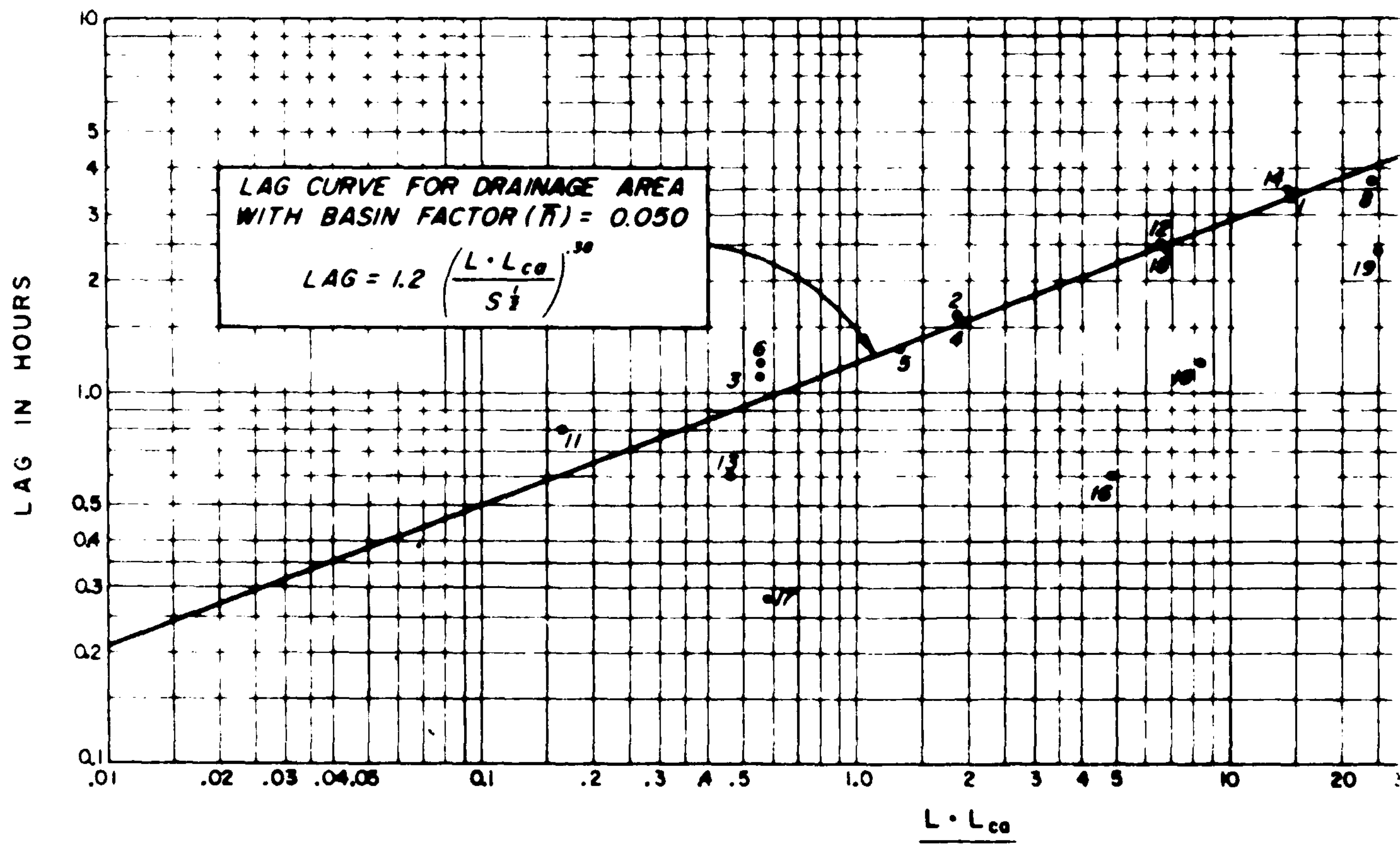


MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

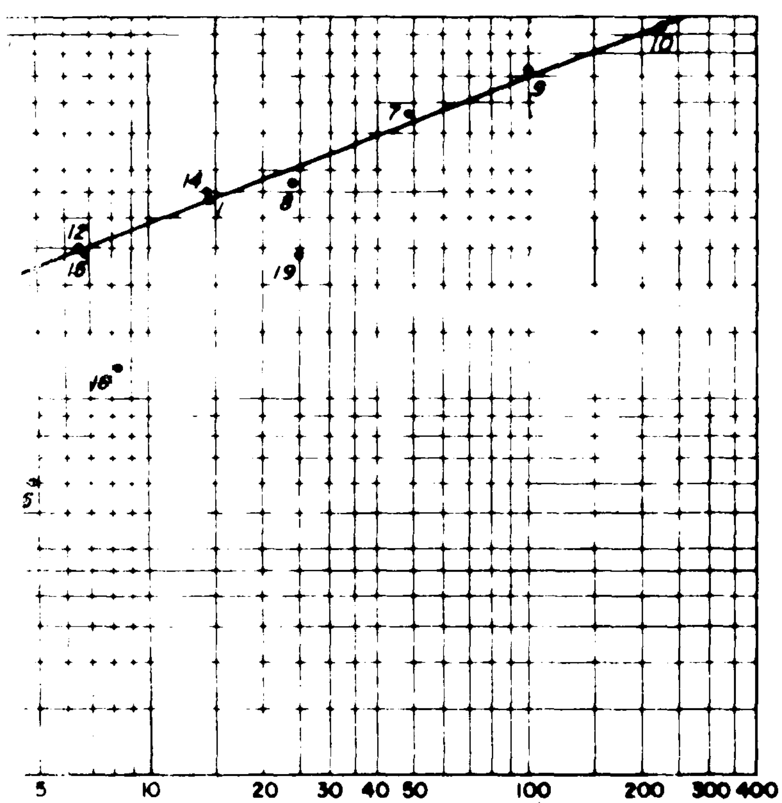
MASS CURVE
STORM OF 24 SEPTEMBER 1939

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

		CONTRIBUTING AREA SQ. MI.	L MILES	L_{ca} MILES	S FT/MI.	LAG HOURS
1.	SAN GABRIEL RIVER AT SAN GABRIEL DAM	162.0	23.2	11.6	350	3.3
2.	WEST FORK SAN GABRIEL RIVER AT COGSWELL DAM	40.4	9.3	4.2	460	1.6
3.	SANTA ANITA CREEK AT SANTA ANITA DAM	10.8	5.8	2.5	690	1.1
4.	SAN DIMAS CREEK AT SAN DIMAS DAM	16.2	8.6	4.8	440	1.5
5.	EATON WASH AT EATON WASH DAM	9.5	7.3	4.4	600	1.3
6.	SAN ANTONIO CREEK NEAR CLAREMONT	16.9	5.9	3.0	1,017	1.2
7.	SANTA CLARA RIVER NEAR SAUGUS	355.0	36.0	15.8	140	5.6
8.	TEMECULA CREEK AT PAUBA CANYON	168.0	26.0	11.3	150	3.7
9.	SANTA MARGARITA RIVER NEAR FALLBROOK	645.0	46.0	22.0	105	7.3
10.	SANTA MARGARITA RIVER AT YSIDORA	740.0	61.2	34.3	85	9.5
11.	LIVE OAK CREEK AT LIVE OAK DAM	2.3	2.9	1.5	700	.8
12.	TUJUNGA CREEK AT BIG TUJUNGA DAM	81.4	15.1	7.3	290	2.5
13.	EAST FULLERTON CREEK AT FULLERTON DAM	3.1	3.2	1.7	140	.6
14.	LOS ANGELES RIVER AT SEPULVEDA DAM	152.0	19.0	9.0	145	3.5
15.	PACOIMA WASH AT PACOIMA DAM	27.8	15.0	8.0	315	2.4
16.	ALHAMBRA WASH ABOVE SHORT STREET	14.0	9.5	4.6	85	.6
17.	BROADWAY DRAIN ABOVE RAYMOND DIKE	2.5	3.4	1.7	100	.28
18.	BALLONA CREEK AT SAWTELLE BLVD.	88.6	11.8	5.6	64	1.2
19.	SAN JOSE CREEK AT WORKMAN MILL ROAD BRIDGE	81.3	23.7	9.1	75	2.4



L_{ca}	S	LAG	ESTIMATED $\bar{n}$
ILES	FT/MI	HOURS	
11.3	350	3.3	0.050
4.2	450	1.6	.050
2.5	690	1.1	.050
4.8	440	1.5	.050
4.4	600	1.3	.050
3.0	1,017	1.2	.055
15.8	140	5.6	.050
11.3	150	3.7	.050
22.0	105	7.3	.055
34.3	85	9.5	.055
1.5	700	.8	.070
7.3	290	2.5	.050
1.7	140	.6	.035
9.0	145	3.5	.050
8.0	315	2.4	.050
4.6	85	.6	.015
1.7	100	.28	.015
5.6	64	1.2	.020
9.1	75	2.4	.030



GUIDE FOR ESTIMATING BASIN FACTOR ($\bar{n}$)

$\bar{n}$=0.200: DRAINAGE AREA HAS COMPARATIVELY UNIFORM SLOPES AND SURFACE CHARACTERISTICS SUCH THAT CHANNELIZATION DOES NOT OCCUR. GROUND COVER CONSISTS OF CULTIVATED CROPS OR SUBSTANTIAL GROWTHS OF GRASS AND FAIRLY DENSE SMALL SHRUBS, CACTI, OR SIMILAR VEGETATION. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$=0.050: DRAINAGE AREA IS QUITE RUGGED, WITH SHARP RIDGES AND NARROW, STEEP CANYONS THROUGH WHICH WATERCOURSES MEANDER AROUND SHARP BENDS, OVER LARGE BOULDERS, AND CONSIDERABLE DEBRIS OBSTRUCTION. THE GROUND COVER, EXCLUDING SMALL AREAS OF ROCK OUTCROPS, INCLUDES MANY TREES AND CONSIDERABLE UNDERBRUSH. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$=0.030: DRAINAGE AREA IS GENERALLY ROLLING, WITH ROUNDED RIDGES AND MODERATE SIDE SLOPES. WATERCOURSES MEANDER IN FAIRLY STRAIGHT, UNIMPROVED CHANNELS WITH SOME BOULDERS AND LODGED DEBRIS. GROUND COVER INCLUDES SCATTERED BRUSH AND GRASSES. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$=0.015: DRAINAGE AREA HAS FAIRLY UNIFORM, GENTLE SLOPES WITH MOST WATERCOURSES EITHER IMPROVED OR ALONG PAVED STREETS. GROUND COVER CONSISTS OF SOME GRASSES WITH APPRECIABLE AREAS DEVELOPED TO THE EXTENT THAT A LARGE PERCENTAGE OF THE AREA IS IMPERVIOUS.

TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE

L_{ca} = LENGTH ALONG LONGEST WATERCOURSE, MEASURED UPSTREAM TO POINT OPPOSITE CENTER OF AREA

S = OVER-ALL SLOPE OF LONGEST WATERCOURSE BETWEEN HEADWATER AND COLLECTION POINT

LAG = ELAPSED TIME FROM BEGINNING OF UNIT PRECIPITATION TO INSTANT THAT SUMMATION HYDROGRAPH REACHES 50% OF ULTIMATE DISCHARGE.

$\bar{n}$ = VISUALLY ESTIMATED MEAN OF THE n (MANNING'S FORMULA) VALUES OF ALL THE CHANNELS WITHIN AN AREA.

NOTE

TO OBTAIN THE LAG (IN HOURS) FOR ANY AREA, MULTIPLY THE LAG OBTAINED FROM THE CURVE BY:

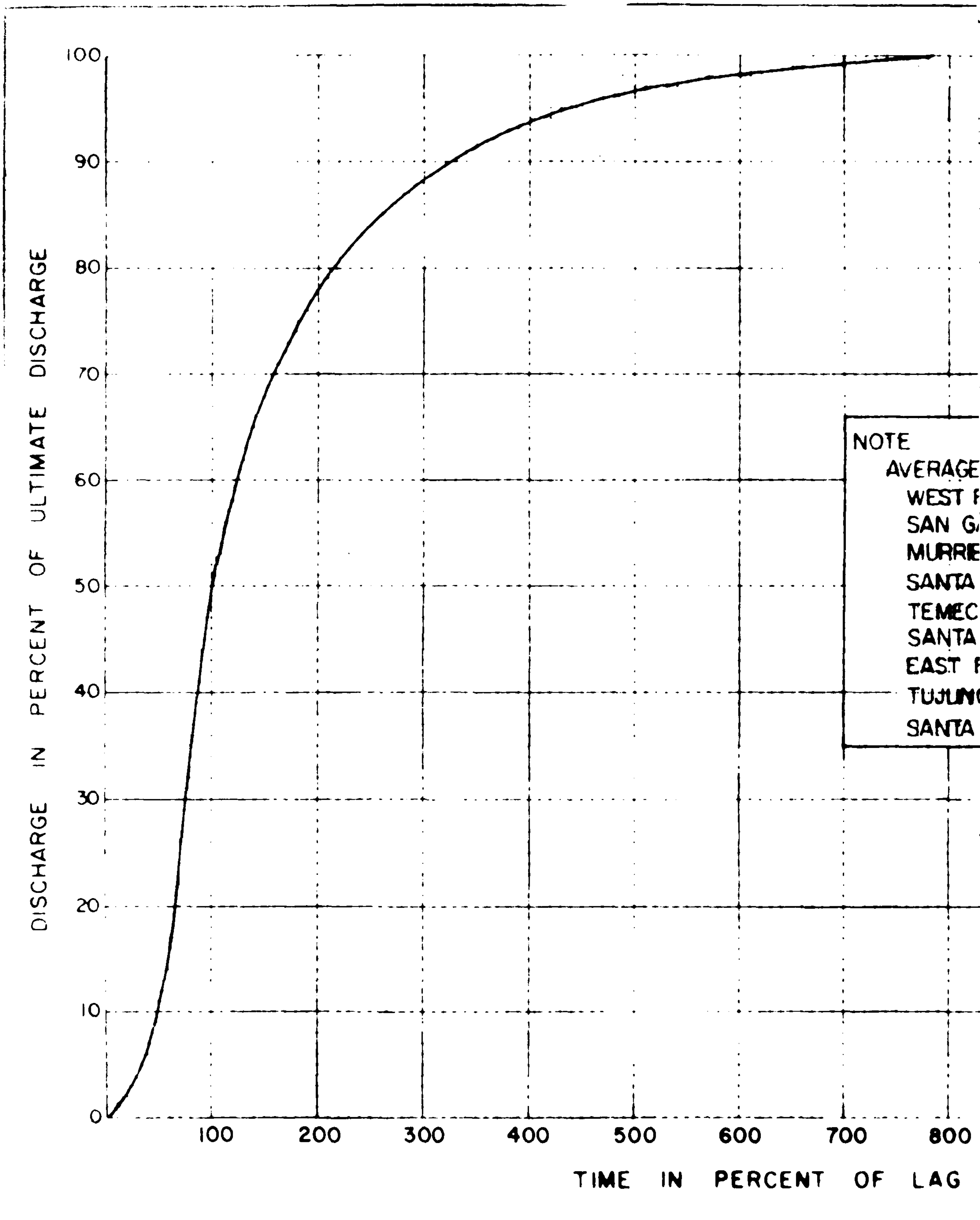
$$\frac{\bar{n}}{0.050} \text{ OR } 20\bar{n}$$

MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

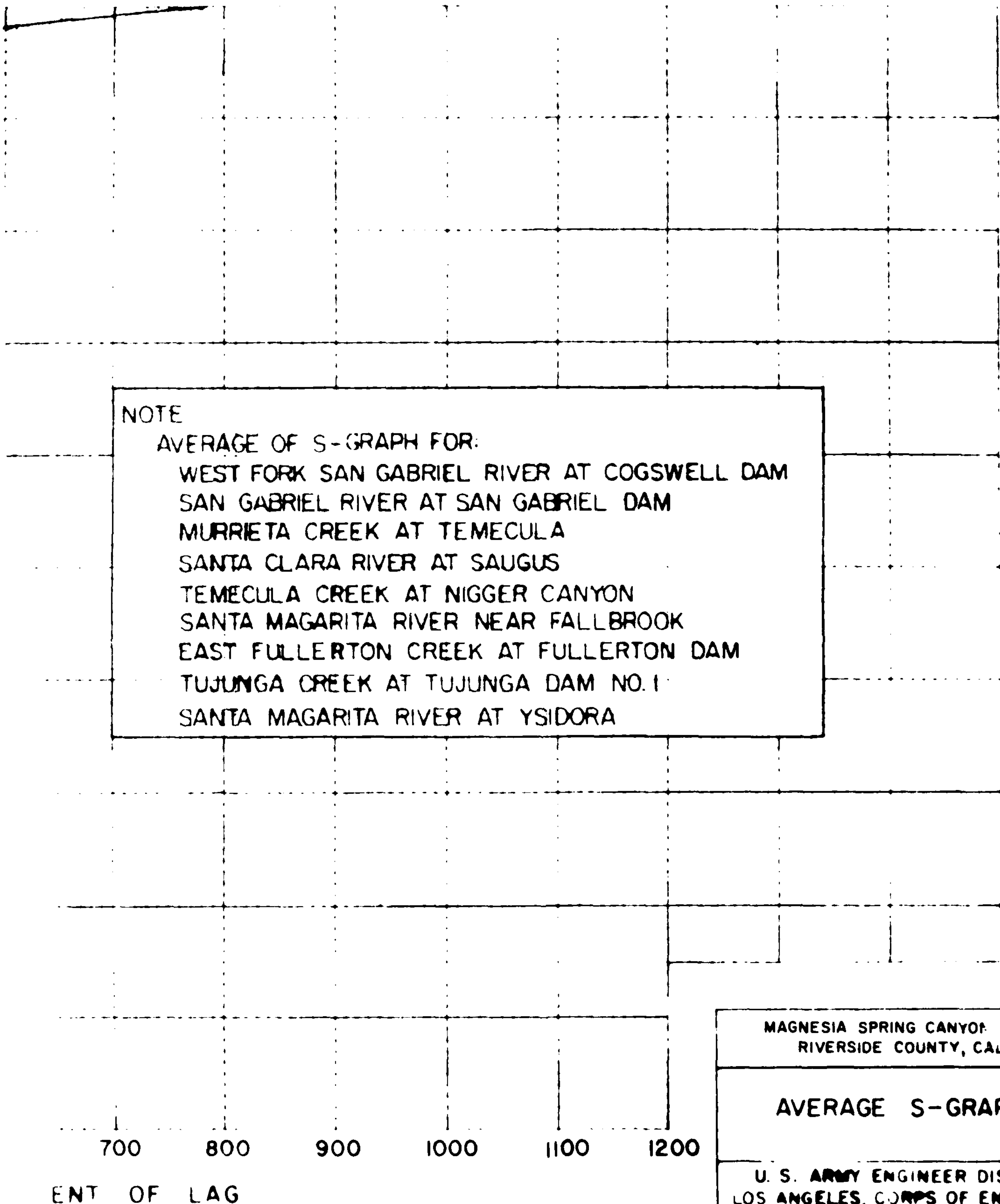
LAG RELATIONSHIPS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEER

11



100
90
80
70
60
50
40
30
20
10
0
DISCHARGE IN PERCENT OF ULTIMATE DISCHARGE
100
200
300
400
500
600
700
800
TIME IN PERCENT OF LAG
NOTE
AVERAGE
WEST F
SAN G
MURRIE
SANTA
TEMEC
SANTA
EAST F
TUJUN
SANTA



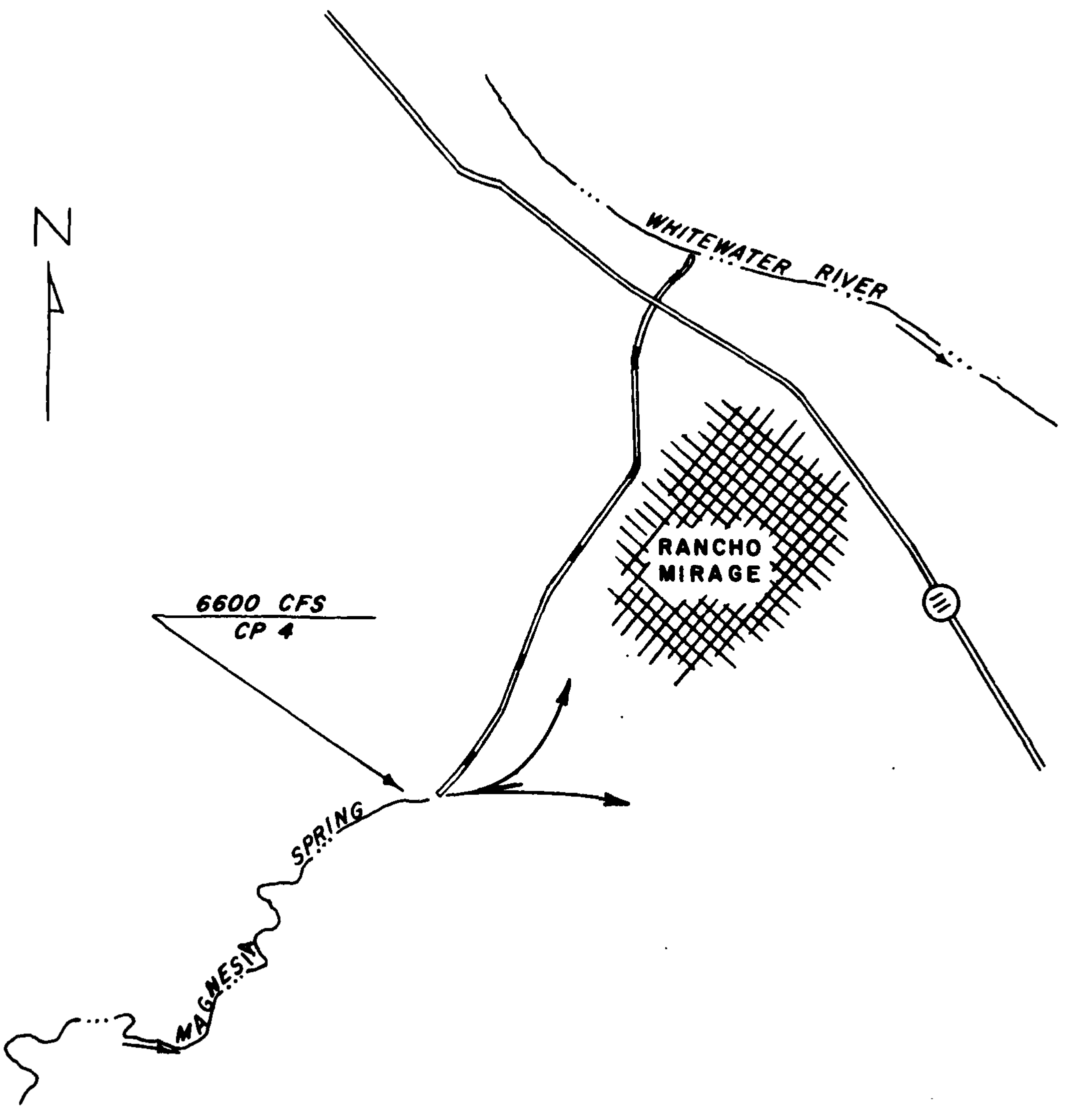
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

AVERAGE S-GRAPH

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE A

II



LEGEND

6600 CFS / CP 4 — SPF PEAK DISCHARGE IN CFS / CONCENTRATION POINT

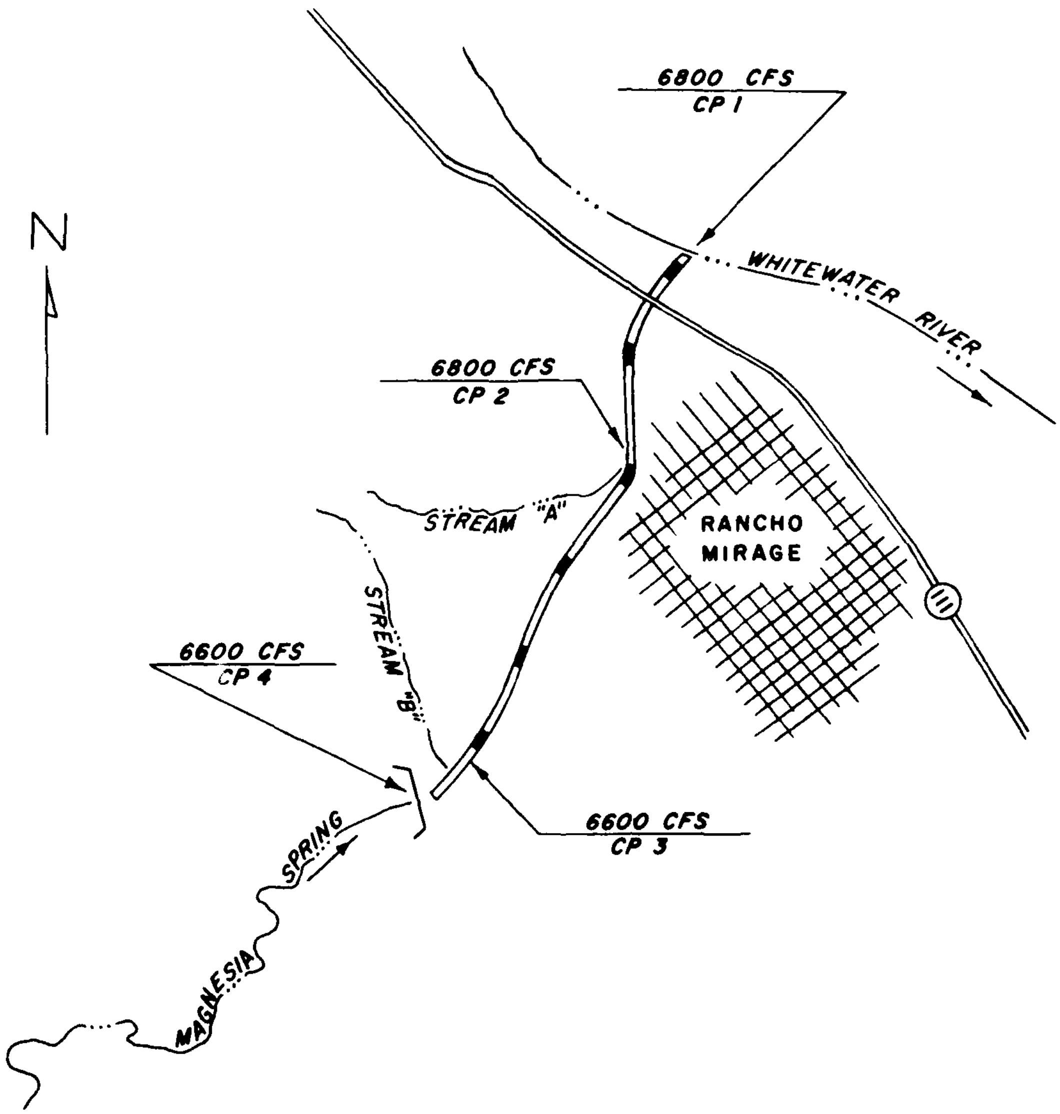
EXISTING CHANNEL

DIRECTION OF FLOW

MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

STANDARD PROJECT FLOOD
PEAK DISCHARGES
WITHOUT PROJECT

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS



LEGEND

6800 CFS / CP 1 — *SPF PEAK DISCHARGE IN CFS / CONCENTRATION POINT*

DEBRIS BASIN SITE

PROPOSED CHANNEL

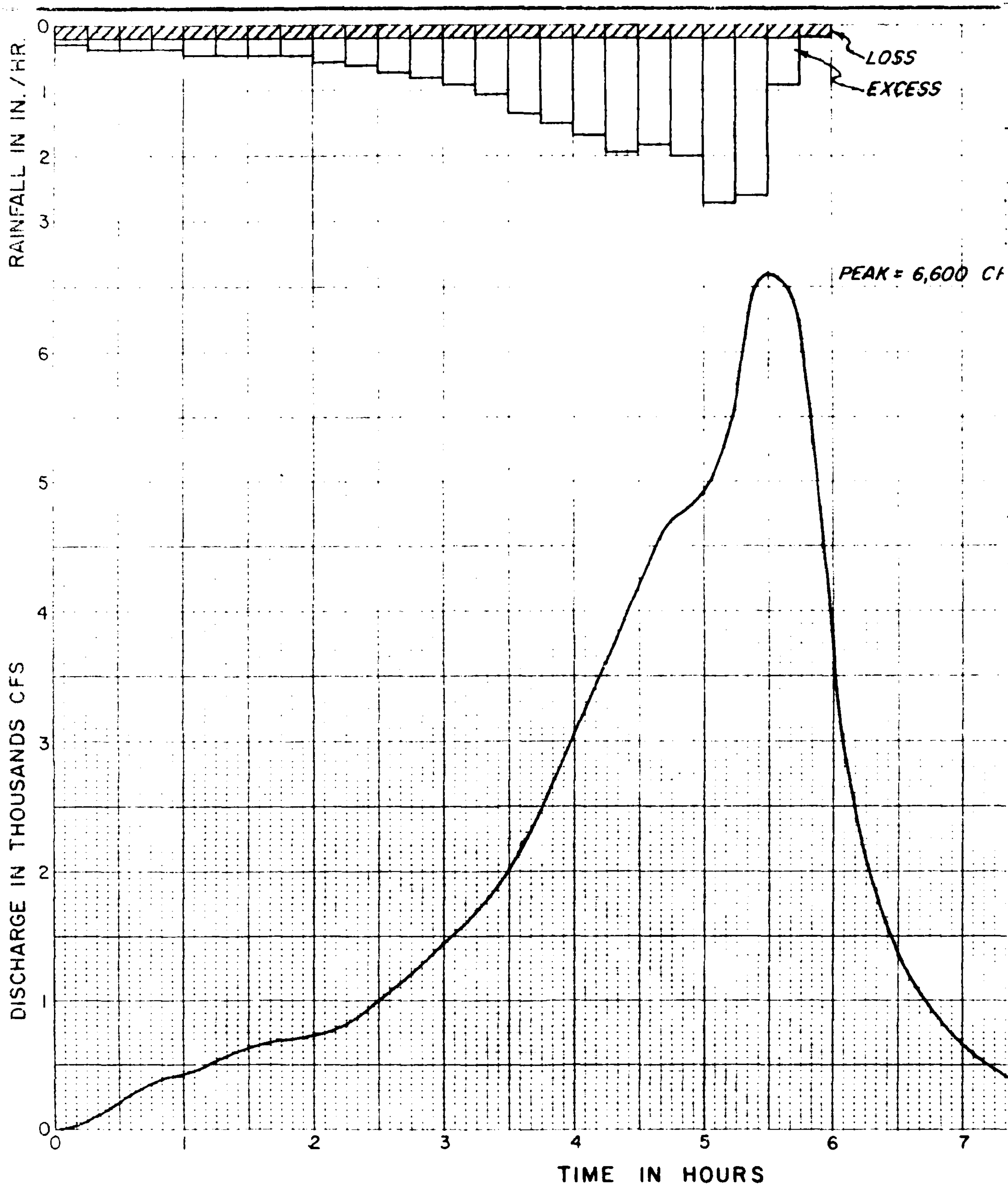
DIRECTION OF FLOW

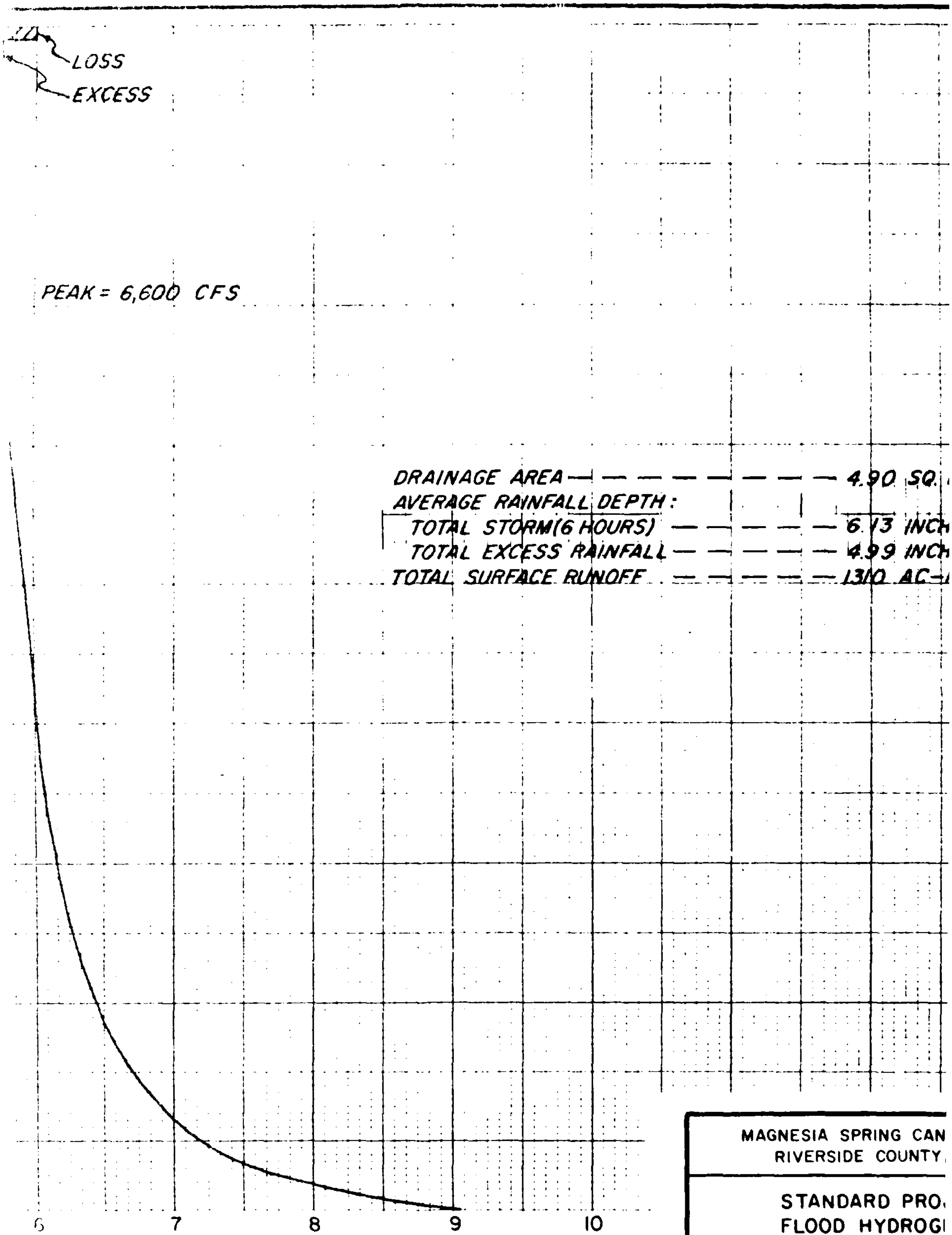
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

**STANDARD PROJECT FLOOD
PEAK DISCHARGES
WITH PROJECT**

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

APPENDIX A


RAINFALL IN IN. / HR.
LOSS
EXCESS
PEAK = 6,600 CF
DISCHARGE IN THOUSANDS CFS
TIME IN HOURS



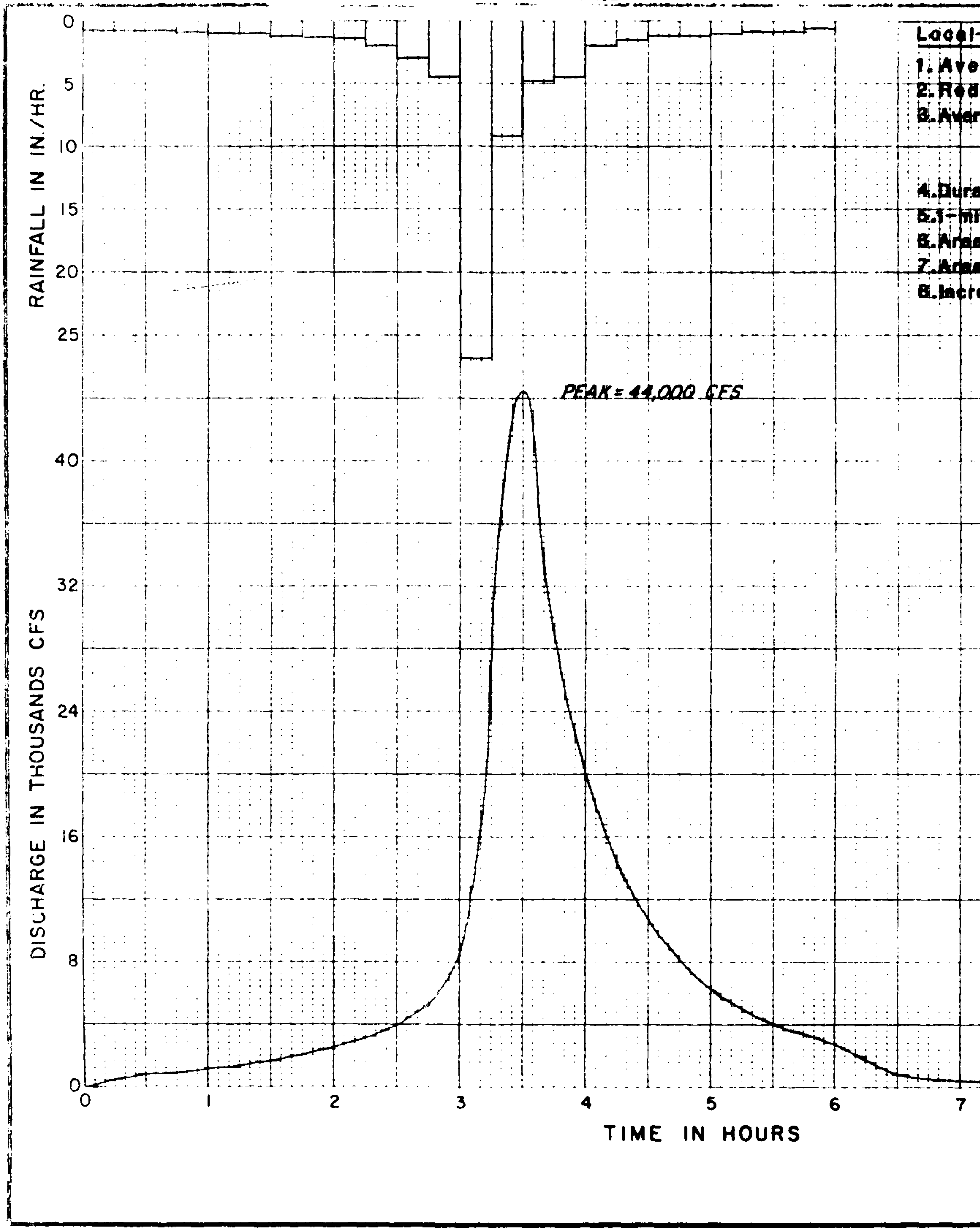
MAGNESIA SPRING CAN
RIVERSIDE COUNTY,

STANDARD PRO.
FLOOD HYDROG

DEBRIS BASIN S
(@ ELEV. 480

U. S. ARMY ENGINEE
LOS ANGELES, CORPS O

II



RAINFALL IN IN./HR.
0
5
10
15
20
25
PEAK = 44,000 CFS
DISCHARGE IN THOUSANDS CFS
40
32
24
16
8
0
0
1
2
3
4
5
6
7
TIME IN HOURS

Local-storm PMP computation:

1. Average 1-hr 1-mi^2 PMP for drainage ---------- 12.5 in.
2. Reduction for elevation ---------- 0 %
3. Average 6/1-hr ratio for drainage ---------- 1.5

	Duration (hr)									
	1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio	63	83	93	100	121	132	140	145	150	%
5. 1-mi^2 PMP for indicated duration	7.9	10.4	11.6	12.5	15.1	16.5	17.5	18.1	18.8	in.
6. Areal reduction	85	87	88	90	92	92.5	93	93.5	94	%
7. Areal reduced PMP	6.7	9.0	10.2	11.3	13.9	15.3	16.3	16.9	17.7	in.
8. Incremental PMP				11.3	2.6	1.4	1.0	0.6	0.8	in.
	6.7	2.3	1.2	1.1						

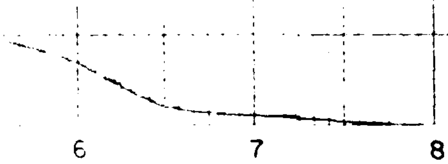
DRAINAGE AREA ---------- 4.90 SQ. MI.

AVERAGE RAINFALL DEPTH:

TOTAL STORM (6 HOURS) ---------- 17.70 INCHES

TOTAL EXCESS RAINFALL ---------- 16.80 INCHES

TOTAL SURFACE RUNOFF ---------- 4390 AC-FT



MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF

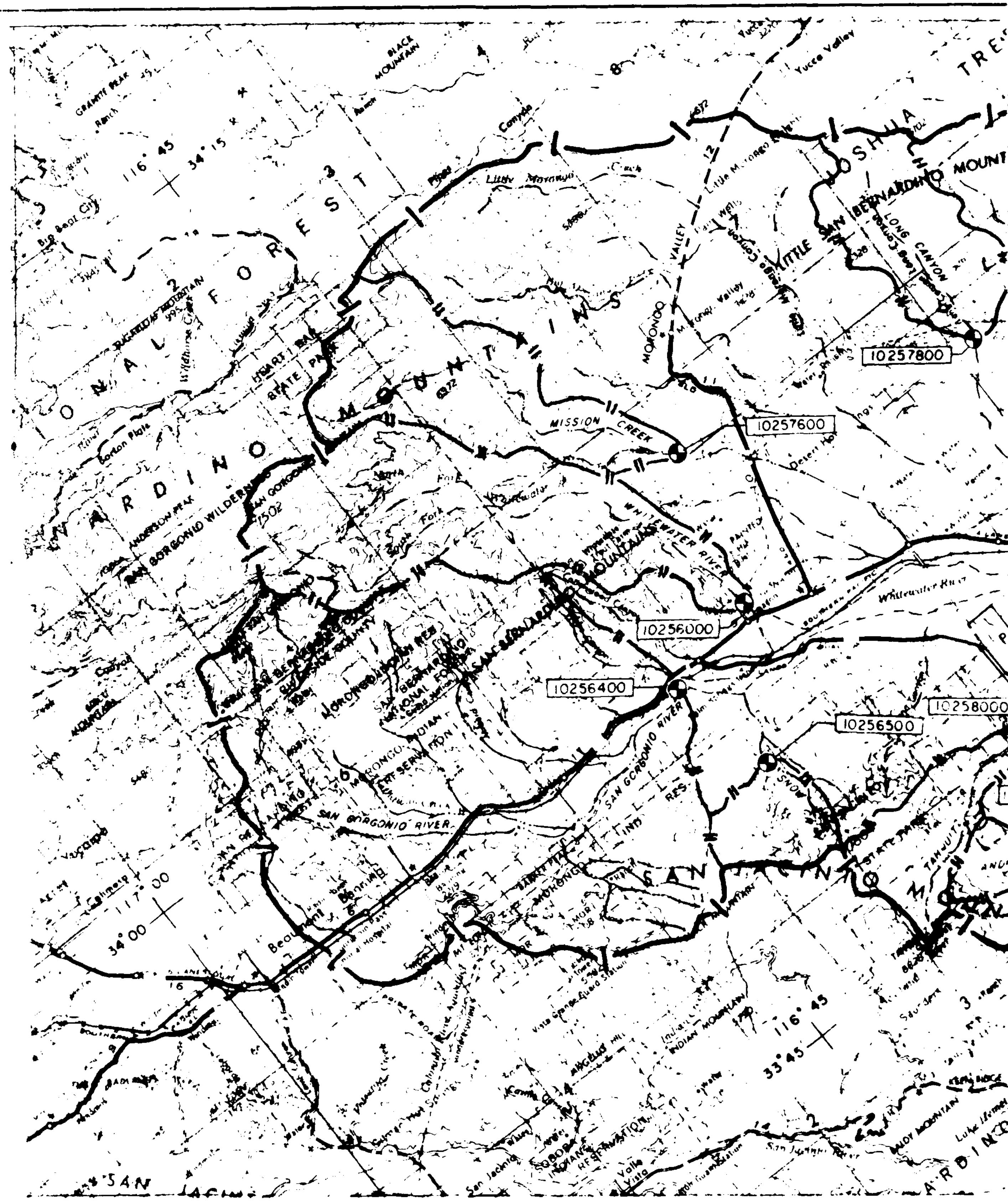
PROBABLE MAXIMUM
FLOOD HYDROGRAPH

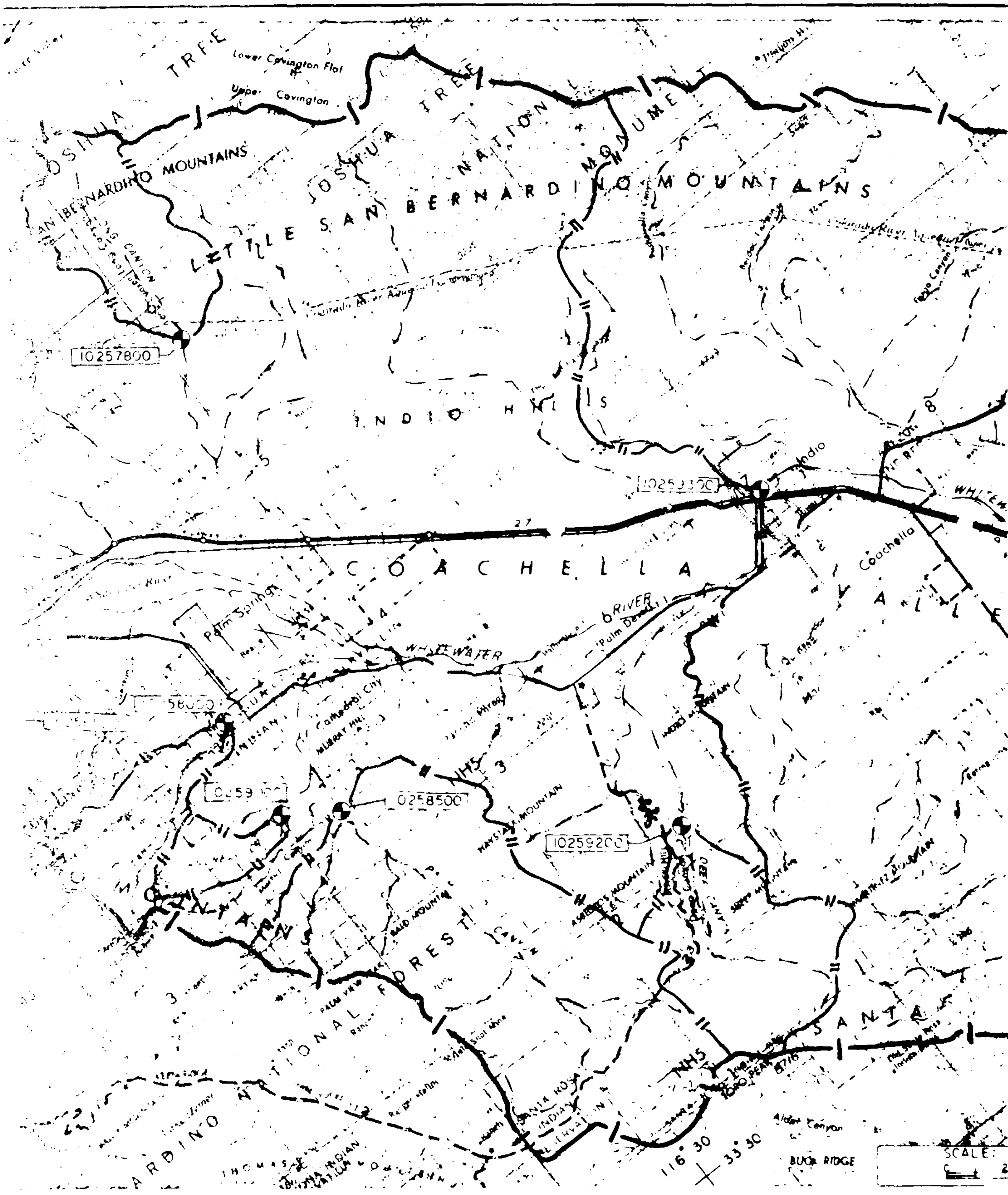
DEBRIS BASIN SITE
(@ ELEV 480 FT)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEER

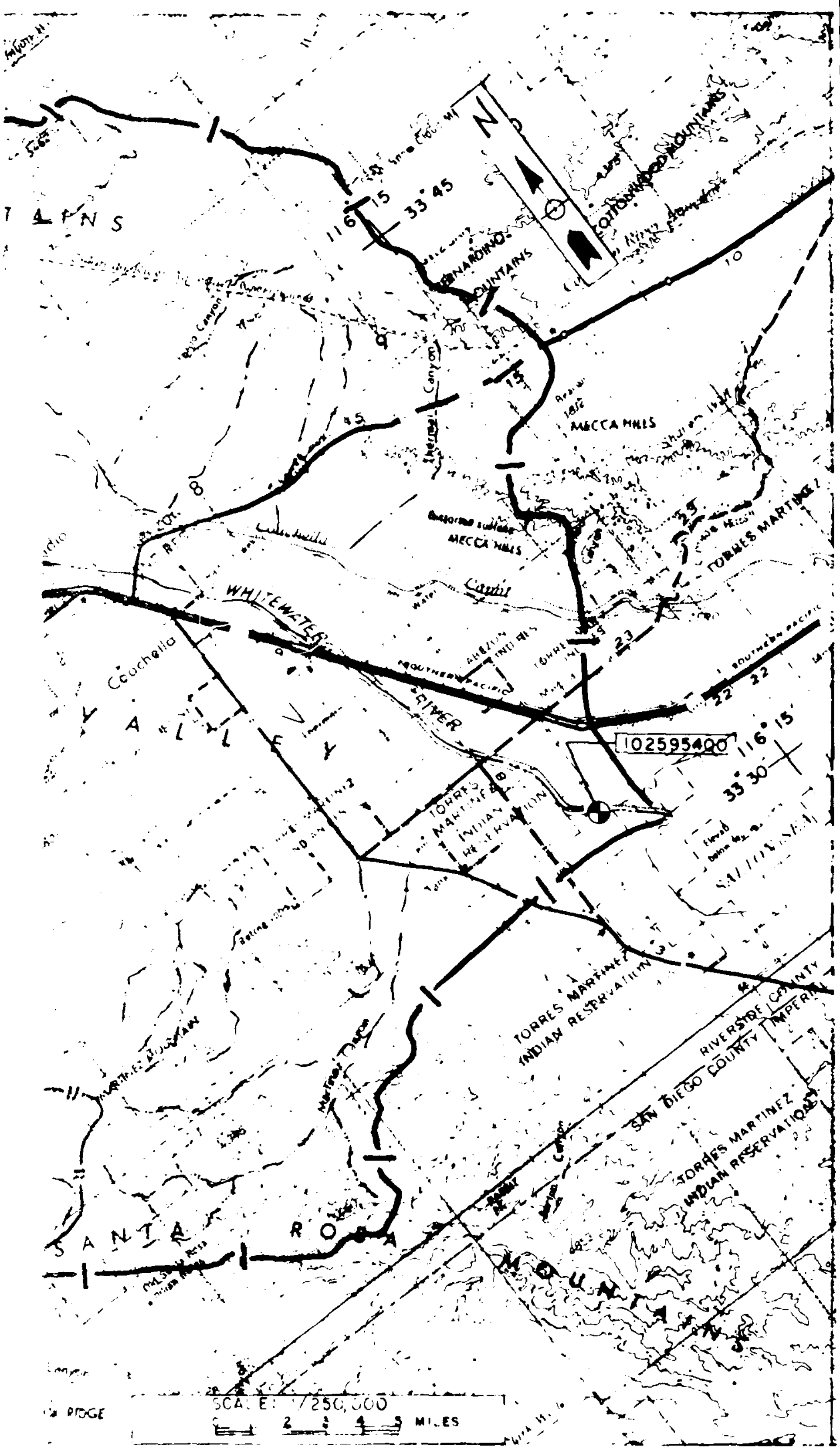
PLATE

II


10257800
10257600
10256000
10256400
10258000
10256500
MISSION CREEK
WHITEWATER RIVER
SAN GORGONIO RIVER
SAN GORGONIO WILDERNESS
LITTLE SAN BERNARDINO MOUNT
LONG CANYON
MORONGO VALLEY
Yucca Valley
MORONGO INDIAN RESERVATION
SAN BERNARDINO NATIONAL FOREST
116° 45
34° 15
117° 00
34° 00
33° 45



Lower Covington Flat
Upper Covington
JOSHUA TREE
LITTLE SAN BERNARDINO MOUNTAINS
JOSHUA TREE NATIONAL MONUMENT
10257800
INDIO HILLS
Indio
10259300
WHITEWATER
COACHELLA
Coachella
VALLEY
Palm Springs
WHITEWATER RIVER
Cathedral City
MURRAY HILL
58000
10259100
10258500
10259200
INDIO MOUNTAIN
HAYSTACK MOUNTAIN
ASBESTOS MOUNTAIN
BALD MOUNTAIN
MARTINEZ MOUNTAIN
FOREST
SAN BERNARDINO NATIONAL FOREST
SANTA ROSA INDIAN RESERVATION
SANTA
TORO PEAK
Alder Canyon
BUCK RIDGE
116 30
33 30
SCALE



LEGEND

- DRAINAGE AREA BOUNDARY FOR WHITEWATER RIVER WATERSHED
- DRAINAGE AREA BOUNDARY FOR WATERSHED ABOVE STREAMGAGING STATION
- USGS STREAMGAGING STATION
- 10257800 USGS NUMBER

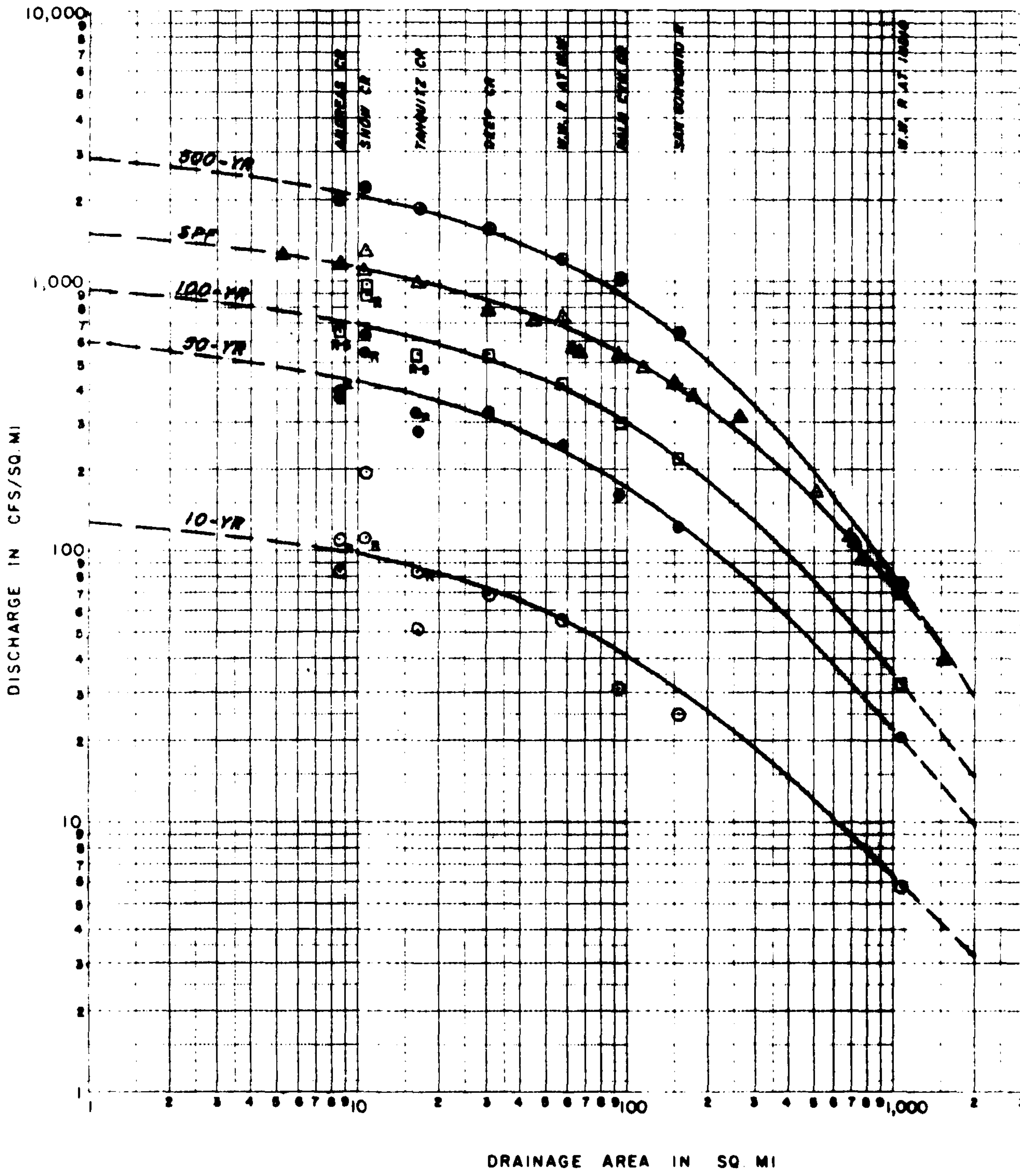
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

STREAM GAGE LOCATIONS

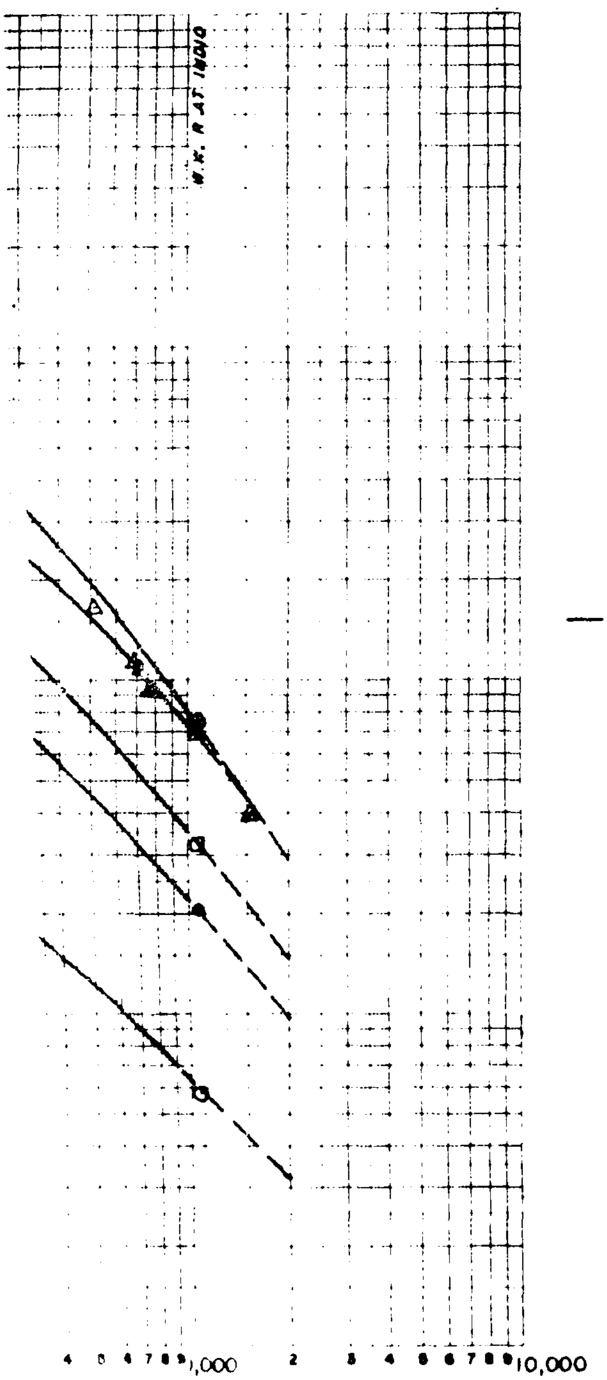
U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE A

JH



10,000
1,000
100
10
1
10
100
1,000
500-YR
SPF
100-YR
50-YR
10-YR
SNOW CR
TAHQUITZ CR
DEEP CR
DISCHARGE IN CFS/SQ MI
DRAINAGE AREA IN SQ MI



LEGEND

- ⊛ *500-YR FLOOD PEAK DISCHARGE*
- △ *SPF PEAK DISCHARGE*
- ⊡ *100-YR FLOOD PEAK DISCHARGE*
- ● *50-YR FLOOD PEAK DISCHARGE*
- ⊙ *10-YR FLOOD PEAK DISCHARGE*
- R *SUBSCRIPT "R" INDICATES PEAK FROM RAINFALL-RUNOFF*
- R-S *SUBSCRIPT "R-S" INDICATES RAINFALL-RUNOFF PEAK SAME AS STATISTICAL PEAK*
- — — — *EXTRAPOLATED CURVE*

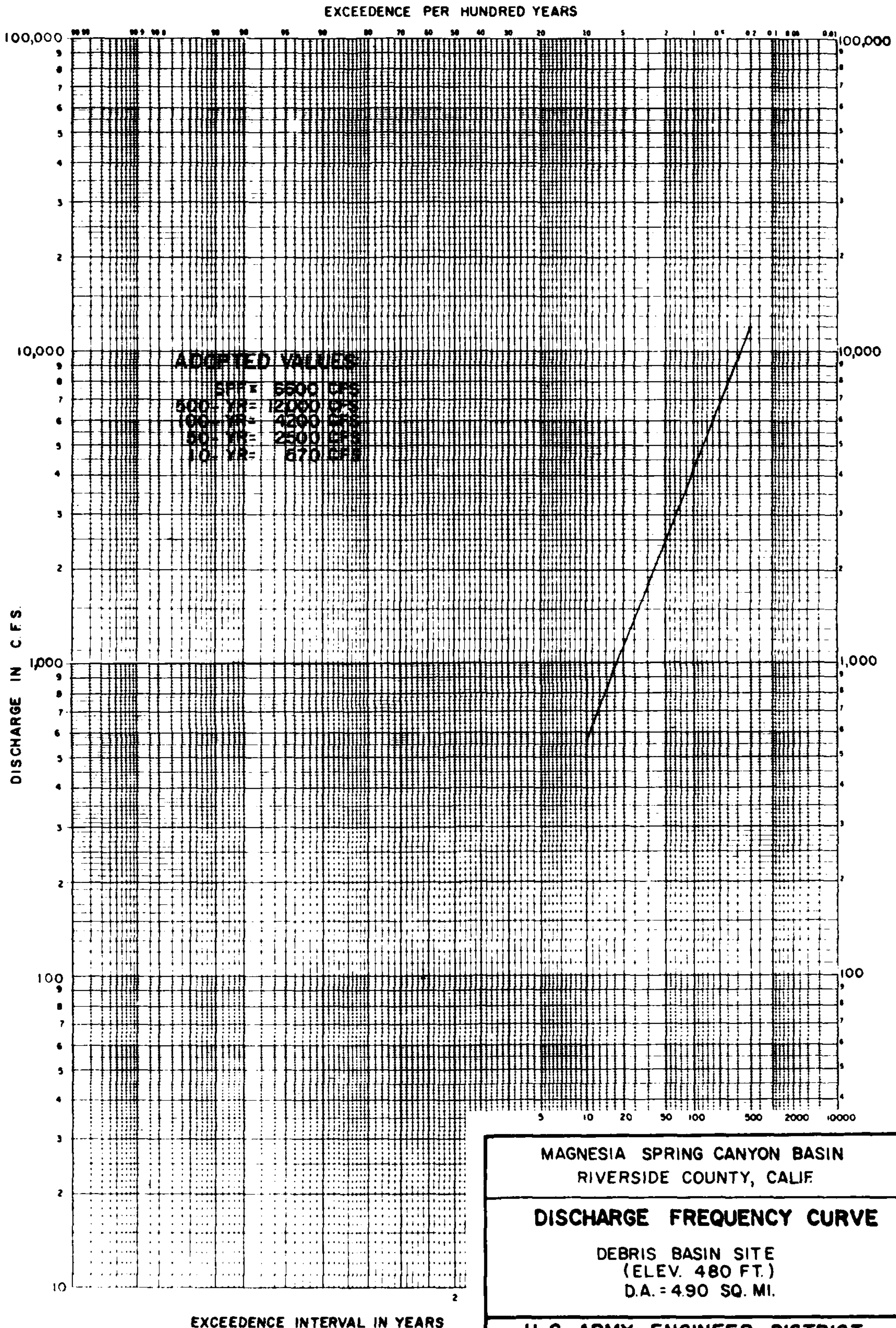
MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

PEAK DISCHARGE
VS.
DRAINAGE AREA CURVES

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE A-

II



MAGNESIA SPRING CANYON BASIN
RIVERSIDE COUNTY, CALIF.

DISCHARGE FREQUENCY CURVE

DEBRIS BASIN SITE
(ELEV. 480 FT.)
D.A. = 4.90 SQ. MI.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE A-15

APPENDIX B

WHITEWATER RIVER BASIN, CALIFORNIA

MAGNESIA SPRING CANYON

DETAILED PROJECT REPORT FOR FLOOD CONTROL

RIVERSIDE COUNTY

APPENDIX B

HYDRAULIC DESIGN

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

SEPTEMBER 1983

Magnesia Spring Canyon
Detailed Project Report for Flood Control
Riverside County, California
Appendix B
Hydraulic Design

Table of Contents

I. INTRODUCTION

1.01 General. In order to safely convey the standard project flood (SPF) through the community of Rancho Mirage, the project would consist of the following major elements: Inlet structure and debris basin; spillway chute and transition; approximately 5,500 feet of concrete-lined rectangular channel; and an outlet energy dissipator. The basis for the design of this project is founded on approved design practice and on theoretical analyis, using applicable criteria set forth in EM 1110-2-1601 Hydrualic Design of Flood Control Channels, EM 1110-2-1603 Hydraulic Design of Spillways, and Hydraulic Design Criteria prepared by the U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

II. DEBRIS BASIN

2.01 General. Because of the sand producing potential of the San Jacinto Mountains, a debris basin would be required at the upstream end of the concrete-lined channel to insure that the inlet capacity is not reduced due to sand deposition during the occurrence of a major storm; to minimize the scour of the concrete lining by coarse sediments being transported at high velocities; and to insure the functional adequacy of the outlet structure. The debris basin would consist of a compacted earth embankment, excavated basin, inlet structure, intake tower, pool drain, and spillway structure. For plan and profile details see plate D-1

2.02 Debris Storage. The criteria for determining the debris volume for the basin is presented in Hydrology Appendix A. From past experience, it has been found that the slope of material deposited after a major flood averages about

one-half of the original natural slope. The capacity of the debris basin (150,000 cubic yards) was determined by calculating the volume between the excavated invert of the basin and the deposition slope projected upstream from the spillway crest at 0.5 of the natural slope. Excavation in the basin is necessary to provide the required debris volume, and to provide material for the embankment. In order to reduce the frequency of maintenance, material brought in by smaller floods could be stored in the basin, provided that not more than 25 percent of the basin capacity is so utilized.

2.03 Upstream Inlet. A lined inlet structure would be provided at the upstream end of the debris basin. This is required in order to stabilize the anticipated streambed degradation upstream from the excavated basin. Specifically, the structure, would stabilize the entire upstream slope face of the basin inlet (approximately 450 feet) with an 18 inch grouted rock cover at a side slope of 1 vertical on 3 horizontal.

2.04 Spillway and Embankment Elevations. The rectangular spillway would be located on the embankment and designed as a broad crest weir to pass the probable maximum flood (PMF) with a peak of 44,000 cubic feet per second (cfs). The spillway crest length of 190 feet and the elevation of 488.0 feet National Geodetic Vertical Datum (NGVD) were found to be the most feasible as indicated by studies of the relationship of debris storage, embankment height, spillway crest length, and spillway transition length. The spillway was rated by assuming critical depth over the crest. Thus, for a discharge of 44,000 cfs, critical depth would be 11.9 feet and the maximum water surface elevation would be at 505.8 NGVD. The top of the embankment at the spillway crest would be at elevation 511.0 feet NGVD providing 5.2 feet of freeboard.

2.05 Spillway Structure. The spillway structure would consist of a short upstream approach channel, a crest section, and a downstream chute. The approach channel (having an adverse slope of 0.020) would be formed by extending the spillway walls at a 1:20 wall flare upstream from the crest section and would intersect with the upstream slope of the embankment. The chute would extend downstream from the spillway crest to a point about 200 feet (Station 77+27.91) downstream from the downstream toe of the embankment The invert slope from the crest would be 0.20870 which is connected by a 25-foot vertical curve to the invert slope of 0.04550. In order to clear a high bluff along the left side, while accommodating an alinement as near to the natural streambed as possible, it was necessary to shorten the structure through the use of a divider wall. This would allow the channel widths to converage at the quicker rate of 1:10 for each wall. The tops of the walls would be based on the PMF of 44,000 cfs with a minimum freeboard of 2 feet.

2.06 Pool Drain. The pool drain would consist of an intake tower located upstream of the spillway with the top of that tower 1 foot above the elevati of the assumed debris level at that point; and a 36-inch, reinforced-concret pipe (RCP) under the embankment with a slope of 0.05584 and invert elevation of 477.20 feet NGVD at the tower and 464.00 feet NGVD at the downstream end where it would enter a junction structure at Station 79+47.91. The junction structure would divert flows from the debris basin to a spreading area approximately 350 feet east of the dam embankment. The drain pipe would operate under inlet control (not under pressure). As such, its discharge capacity would range between 40 cfs with the water surface at the soffit of the intake tower pipe and 120 cfs with the water surface at spillway crest elevation. The pool would drain within one day.

2.07 Diversion Drain. Working in conjunction with the pool drain is a diversion drain. Flows from the pool drain would enter a junction structure at Station 79+47.91 from which 4 drain pipes would exit. Two of the drains would serve to divert no more than a total of 50 cfs to a spreading area east of the dam embankment. These flows, operating under inlet control, would pass through a flow restrictor and would exit two 36" RCP's at an approximate elevation of 460.0 feet NGVD. Both pipes would be between 300 and 350 feet in length and would have approximate slopes of 0.0133 to 0.0144. The exact location of each terminus will be coordinated with local interests at a later date. The remaining 2 pipes would be directed back to the right channel spillway. One of the pipes, a 36" RCP, would serve as the primary drain to the spillway channel. The second pipe, a 48" RCP, would function as an emergency drain should the other three pipes become inoperable. Because of the need for maintenance and emergency shutdowns, both the diversion and primary drains would be gated. However, under normal operational conditions, all of the drains would be in the fully opened position.

III. CHANNEL

3.01 General. The channel from the downstream end (station 77+29.91) of the spillway chute to the beginning of the confluence with stream "A" (station 37+00.00) was designed for the SPF peak discharge of 6,600 cfs. The channel downstream of the confluence to the end of the project was designed, for 6,800 cfs. Elements pertaining to the hydraulic design of the channel from Station 71+62.09 to Station 16+16.67 are discussed in the following paragraphs.

3.02 Alinement. The proposed channel would follow generally along the alinement of the existing channel. It would contain five curves with deflection angles ranging from 16° 58'23" to 44° 05'23". The radii of the circular curves would vary from 600 to 900 feet with upstream and downstream spiral transitions.

3.03 Gradient. Invert grades were selected to avoid excessive channel excavation and to maintain stable supercritical flow except in the downstream energy dissipator. The slope of the channel invert would range from a maximum of 0.04096 to a minumum of 0.02809.

3.04 Cross-section. The channel would be concrete-lined and rectangular in cross-section with a uniform bottom width of 20 feet. The invert of the curved reaches would be superelevated.

3.05 Transitions. The transition from the spillway to the channel would be approximately 565 feet in length and because of the continuance of the divider wall, have a convergence on each of the 4 walls of 1 foot in 10 feet. There would be a 267-foot transition at the downstream end of the concrete channel with the base width diverging 1 foot in 20 feet.

3.06 Side Drainage. There are two subareas that contribute major side inflow to the channel. Both enter from the left side. Stream "A" would contribute 500 cfs between Station 36+60.00 and Station 37+60.00. Its flows would be introduced to the channel via a 100 foot wide side overflow spillway. Stream "B" would contribute 300 cfs betwee Station 66+35 and Station 67+50. Its flows are slightly less concentrated and would be introduced to the channel via a 115 foot wide side overflow spillway.

3.07 Water-Surface Computations. The water-surface profile for the design discharges was determined by the reach method based on the Manning's formula. A Manning's roughness coefficient,"n" of 0.014 was used for the design of the channel while 0.012 was used for velocity consideration. The "n" values used and the equivalent "K" values based on plate 4 of EM 1110-2-1601 are shown in the following tabulation:

Design Item	R	K (ft.)	n
Discharge capacity	5.61	0.00330	0.014
	4.25	0.00370	0.014
Maximum velocity	5.12	0.00068	0.012
	4.05	0.00075	0.012

For n=0.014, depths of flow (including entrained air) and velocities would range between 7.4 and 12.8 feet and between 25.7 and 51.7 fps respectively. For n=0.012, depths and velocities would range between 6.8 and 10.5 feet and between 31.5 and 57.3 fps respectively. Air entrainment was considered as being additive to nonaerated flow depths. For the project, this increase ranged from 0.16 feet to 0.85 feet. These increments were determined using the design curve for air entrainment on page III-47 of EM 1110-2-1601.

3.08 Superelevation. Superelevation of the transverse water surface was computed for the two ranges of "n" values and, when added to the corresponding water depth, it was found that the design based on "n" = 0.014 should be used in the curved reaches. For the five curve locations, superelevation (Y) would range from 0.83 to 0.97 feet. And, since 2 Y was greater than 0.5 feet, the channel inverts at these locations were also banked.

3.09 Bridges. There is only one bridge in the project reach. It is at Route 111. Design considerations were required because of constraints introduced by the invert grade slope, the center pier and its footing elevation. The channel design through the bridge was based on the Kock-Cartanjen momentum equation 16, EM 1110-2-1601. In the design, the concrete circular row of piers were converted to that of a single concrete diaphram and an upstream rounded sloping pier nose extension attached. Two foot of additional blockage (due to potential debris) was assumed on each side of the pier.

3.10 Freeboard. The minimum freeboard would be 2 feet for the rectangular channel. Bank protection for the Whitewater River in the vicinity of the confluence would have a minimum freeboard of 2.5 feet.

IV. DOWNSTREAM ENERGY DISSIPATOR

4.01 General. The outlet energy dissipator's primary function is to reduce the incoming high velocity flows down to a rate at which a hydraulic jump would be forced in the transition. A backwater condition would begin with critical control at the downstream end. Further, a hydraulic jump was also investigated for coincidental flows occurring on the Whitewater River. The outlet energy dissipating structure would consist of the following features: (a) a rectangular channel transistion from a 20-foot base width at Station 16+16.67 to a 46.67 foot base width at Station 14+00.00, (b) a reach of rectangular channel with a base width of 46.67 feet from Station 14+00.00 to Station 11+50.00 (downstream end), (c) velocity reduction impact blocks ranging in size from 1' wide by 1' high to 3'wide by 3' high between Stations 16+00.00 and 13+50.00 and, (d) a higher backwater inducing group of 3'wide by 4' high impact blocks between Stations 13+50.00 and 11+50.00. For details of shape, spacing, number of rows and location of baffle blocks see plate D-5.

4.02 Block Sizing. The analysis for the energy dissipator was based on a design that was patterned after that used in the Los Angeles District Corps of Engineer Design Memorandum No. 4 for Santa Paula Channel dated March 1972. Using the design relationship of "n" value and corresponding block size determined for the Santa Paula Creek Channel design, the same relationship was also incorporated into this design. However, at the upstream end where the 2' wide by 2' high blocks were initially used, incomplete submergence would generate excessive splash and turbulence. This necessitated that these blocks be redesigned in this area. Consequently, using a Froude number and block height relationship developed again from the Santa Paula Channel design, 1' wide by 1' high "A" type blocks were found to function adequately. Final block size and corresponding "n" values used in this design were as follows:

Location (Stationing, ft.)	Block Type	Block Size (width x height, ft.)	n
16+00.00 to 14+00.00	A	1x1	0.022
14+00.00 to 13+50.00	B	3x3	0.035
13+50.00 to 11+50.00	C	3x4	0.045

4.03 Block Arrangement. The blocks were arranged in a pattern recommended in the Santa Paula Channel design. This pattern would enhance their function as impact devices instead of as a roughness element.

4.04 Water Surface Computations. Water surface profiles were computed for the energy dissipator using the "n" values cited above in Paragraph 4.02 for two different inflow conditions; (1) transitions from 20 foot wide channel with n=0.014 and (2) transition from 20 foot wide channel with n=0.012. In

addition, a water surface profile was developed for a smaller discharge. Under all cited conditions a hydraulic jump would be induced within the outlet structure, and critical depth control would be maintained at the downstream end. However, for major coincidental flows at the confluence, the Whitewater River would generate a backwater condition on Magnesia Springs which would override the critical depth control at the end sill and drown out the effects of the hydraulic jump in the stilling basin. Finally, since the combination of the coincidental discharges would never exceed the Whitewater River SPF peak discharge of 78,000 cfs, the Whitewater SPF was used as the controlling flood in governing the height of the outlet channel. Plate D-4 illustrates the water surface profile for the West Magnesia Springs SPF discharge of 6,800 cfs as controlled by critical depth at the end sill.

V. WEST MAGNESIA SPRINGS - WHITEWATER RIVER CONFLUENCE

5.01 General. In order to maintain functional intergrity of the outlet structure from flows in the Whitewater River and minimize damages to the Whitewater River banks because of existing West Magnesia Springs flows, bank stabilization was required in this area.

5.02 Bank Stabilization Sizing. Bank stabilization was designed following criteria set forth in the 14 May 1971 ETL 1110-2-120; "Additional Guidance for Riprap Channel Protection." 1 vertical on 3 horizontal slope protection was designed to withstand a maximum channel flow velocity of 17 fps.

5.03 Scour and Wave Action Considerations. Toes of the bank protection were extended 10 feet below natural invert grade to safeguard against the threat of excessive streambed degradation. However, this was further increased by

10 feet for a reach of bank immediately opposite the outlet structure where additional scour could be anticipated. Finally, the bank protection was extended far enough downstream on both sides to account for any potentially damaging wave actions, generated by the confluence, to dampen out.

APPENDIX C

WHITEWATER RIVER BASIN, CALIFORNIA

MAGNESIA SPRING CANYON

DETAILED PROJECT REPORT FOR FLOOD CONTROL

RIVERSIDE COUNTY

APPENDIX C

GEOLOGY AND SOILS

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

SEPTEMBER 1983

APPENDIX C

GEOLOGY, SOILS AND MATERIALS

RANCHO MIRAGE

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APPENDIX C

GEOLOGY, SOILS AND MATERIALS

RANCHO MIRAGE

INTRODUCTION

1. Purpose and Scope. Geotechnical investigations were conducted to determine the extent, distribution and physical properties of the rock and soils at the site of the proposed debris basin and channel, and to obtain detailed information on the foundation, construction materials and groundwater conditions. This appendix describes the geotechnical investigations, testing, seismicity, foundation conditions, methods of analysis, design values, foundation treatments, embankment and channel design, and construction procedures.

2. Description of Project Features. The proposed flood control improvements at Magnesia Spring Canyon at Rancho Mirage, California consist of a debris basin and a channel. The debris basin embankment would be a compacted, homogeneous earthfill structure, approximately 35 feet high and 750 feet long. A concrete-lined broad-crested spillway capable of discharging a maximum probable flood would be built on the embankment. An access road would be provided to the top of the embankment and to the basin area for inspection and maintenance purposes. A grouted stone inlet structure would be constructed at the upstream end of the excavated deb s basin, approximately 600 feet from the embankment centerline. The outlet channel would be designed to convey the standard project floodwaters to the Whitewater River, a distance of approximately 1.4 miles. The channel would be an entrenched concrete-lined rectangular section, 20 feet wide and typically 8 to 10 feet deep.

TOPOGRAPHY AND GEOLOGY

3. Regional Topography and Geology. Magnesia Spring Canyon is located on the northeast side of the Santa Rosa Mountains, see plate C-1. The Santa Rosa Mountains in conjunction with the San Jacinto Mountains form the eastern-most portion of the north-northwest trending Peninsular Ranges. The San Jacinto Mountains, which are separated from the Santa Rosa Mountains by the north-northeast trending Palm Canyon fault, see plate C-2, comprise a late Cretaceous granitic terrain reaching an elevation of 10,805 feet at the summit of San Jacinto Peak. The Santa Rosa Mountains, to the southeast of the San Jacinto Mountains, reach a maximum elevation of 8,716 feet at the summit of Toro Peak. These mountains are granitics (primarily granodiorites and quartz monzonites) of probable late Cretaceous age that had intruded shallow water pre-Cretaceous marine sediments which were metamorphosed into the Palm Canyon Complex.

The metasediments of the Palm Canyon Complex, designated as "ms" on plate C-3, consist primarily of crystalline limestones, marbles, quartzites and mica schists which have been intruded by dioritic and granitic material and injected by pegmatite and primary quartz dikes both parallel and transverse to original bedding. Miller (1944) has described extensive intrusions and localized partial melting by acidic magmas of the metadiorites, biotitic quartzites and schists which have produced sills of banded gneisses.

To the east of the steep eastern face of the San Jacinto-Santa Rosa Mountains is the pronounced topographic low of the Coachella Valley which reaches minus 235 feet at the Salton Sea. The Coachella Valley is an elongated, fault controlled structural basin presently undergoing east-west extension. It is the northwest extremity of the Salton Trough of the Gulf of

California. Alluvium within the valley ranges from semi-consolidated to unconsolidated silts, sands and gravels reaching a depth of 12,000 feet near the San Andreas fault zone on the eastern side of the valley. The major drainage through the valley is the southeast flowing intermittant Whitewater River which terminates in the Salton Sea sink.

Regional structure of the area is controlled by the active San Andreas and San Jacinto fault systems. The San Andreas fault zone forms the boundary between the Little San Bernardino Mountains of the Transverse Ranges to the northeast and the Coachella Valley. The San Jacinto fault zone forms the boundary along the western portion of the San Jacinto-Santa Rosa Mountains. Both of these zones trend roughly parallel to each other in a northwest-southeast direction. Most of the smaller faults, such as the Toro Canyon and Oasis faults, parallel major fault trends.

4. Local Topography and Geology. The proposed debris basin is located across the head of the Magnesia Spring Canyon alluvial fan, approximately 0.6 miles upstream from the edge of the residential community of Rancho Mirage. The gradient of the fan surface in this reach is 3 percent. At the debris basin site the fan is somewhat constricted by bedrock outcrops and is only 550 feet wide. Just downstream from the proposed alinement, it fans out rapidly to a width of one mile through the community of Rancho Mirage. Upstream, the canyon is very narrow where Magnesia Spring issues from the precipitous metasedimentary basement rocks of the Palm Canyon Complex. Within the site area, this formation consists largely of interbedded schists, gneisses, limestones and marbles which have been injected by relatively thin quartz veins parallel to bedding. Schistosity parallels bedding.

Three Quaternary deposits were recognized and mapped at the site: 1) Qt, older terrace deposits which represent relict fan surfaces from Magnesia Spring Canyon; 2) Qf/Qcl, undifferentiated slope wash and tributary alluvial fan deposits; and 3) Qal, recent channel deposits on the active Magnesia Spring Canyon alluvial fan. The site geology map and specific unit descriptions are shown on plate C-4. In addition to the unconsolidated to semiconsolidated recent sediments and the basement rocks, remnants of older fan surfaces exist on both sides of the canyon. The older fan terraces consist of crudely layered, unconsolidated gravelly sands and sands with occasional cobbles and small boulders. Alluvial fans have also developed on small tributary drainages to Magnesia Spring Canyon. The tributary alluvial fans are more poorly layered and sorted than the terrace material, with boulders to 4-foot maximum diameter.

5. Groundwater. The Coachella Valley groundwater basin is divided into four subbasins and four subareas. Magnesia Spring Canyon is within the Thermal Subarea of the Indio Subbasin which is bounded on the northeast by the San Andreas and related faults and on the southwest by the Santa Rosa and San Jacinto Mountains. The overall groundwater gradient is to the southeast towards the Salton Sea. Both surface runoff and semi-perched groundwater discharge into the Salton Sea while groundwater moves beneath the sea through deeper aquifers which extend farther to the southeast. The Palm Springs Subarea to the northwest provides most of the recharge to the basin. The relatively coarse material in the alluvial fans at the base of the Santa Rosa Mountains, such as at Rancho Mirage, also act as small recharge areas.

The Coachella Valley Water District is responsible for monitoring water levels within the Rancho Mirage area. All of the wells monitored are located at least 2.5 miles north of the site. Well measurements taken in October, 1980 from well number T5S R5E 13A1 show groundwater levels averaging 170 feet beneath the ground surface. Overall trends since 1970 indicate the groundwater table is dropping on the average of 1 to 4 feet per year, indicating withdrawal in excess of recharge in this area.

One quarter mile upstream from the proposed embankment, water from the perennial Magnesia Spring sinks rapidly into the coarse channel sands almost as soon as it leaves the bedrock at the canyon head. It apparently then closely follows the bedrock/alluvium contact at depth. Groundwater was not encountered during explorations along the proposed embankment alignment. Test pits 80-1, 80-3 and 80-4 were 20 feet deep and extended down to elevation 450. Drill hole D-2 at the left abutment was rotary drilled to bedrock (elevation 430.4) without any indications of groundwater even though refractive seismic line 80-5 had previously indicated the possibility of groundwater at a depth of approximately 20 feet (elevation 458±). Other than in refractive seismic survey line 80-5 and line 80-1 (1000 feet upstream from the alignment), velocities indicative of saturated alluvium (approximately 5000 fps) were not detected during the geophysical investigations. However, a thin saturated zone on top of the high velocity bedrock would be difficult to detect. Furthermore, incompletely saturated but wet sands will not generate velocities indicative of groundwater. Nonetheless, it is believed that other than during periods of high runoff from the canyons upstream, groundwater will not be encountered during shallow construction excavations.

SEISMICITY

6. Regional Seismicity. The site for the debris basin is located in a highly seismic region dominated by activity along the San Andreas and San Jacinto fault zones, located 8 and 16 miles, respectively, from the site. Major faults and fault zones within a 100-mile radius of the site are presented on plate C-4 along with locations of earthquake epicenters of Magnitude 4+ that have occurred in the years 1932 to 1979. The magnitude of the maximum credible earthquake and maximum bedrock acceleration at the site, fault length and closest distance to the site for the major faults are listed in TABLE C-1.

Since 1932, when instrumented records of earthquakes in Southern California began, a total of eight Magnitude 6+ events have occurred within 100 miles of the project site. These events, along with the 1918 San Jacinto earthquake are listed in TABLE C-2. All of these, except the 1933 and 1947 events, are earthquakes related to movements on either the San Andreas or San Jacinto faults. In addition, the historic San Jacinto earthquake of 21 April 1918 was centered approximately 29 miles from the site. The estimated magnitude of this event was 6.8, based upon a reported intensity of IX to X (Modified Mercalli Scale) at the town of San Jacinto. Since 1933, 102 earthquakes of magnitude 4.0 or greater have occurred within a 25-mile radius of the project site. Ninety-two of these earthquakes were 4.0 to 4.9 magnitudes, most of which were clustered in the general vicinity of the San Andreas and San Jacinto fault zones. Earthquakes of 5.0+ magnitude within 25 miles of the site are listed in TABLE C-3.

7. Site Seismicity. The Oasis, Toro Canyon and Palm Canyon faults, as well as numerous small faults, lie within 10 miles of the debris basin site. From aerial photographs, these faults appear to lie entirely within the pre-Cretaceous metasediments. The unnamed smaller faults lie primarily within bedding planes which indicate they were active during the initial regional uplift of the Santa Rosa Mountains. The Oasis, Toro Canyon and Palm Canyon faults appear to have formed in response to the thrusting of the late Cretaceous granites along the Santa Rosa thrust fault, part of which lies approximately 1 mile directly north of the site. In no areas do any of these faults displace recent deposits and therefore are considered inactive.

In the period of 1933 to 1979 there have been 18 earthquakes with epicenters within 10 miles of the debris basin. The magnitudes range from 2.0 to 4.0 with four falling within the 3.5 to 4.0 magnitude range. None of the epicenters can be correlated to any specific active fault and the ground motions associated with them are smaller and of a shorter duration than motions that would be generated by possible large events several miles away.

8. Fault Hazards. None of these faults lie within the immediate vicinity of the debris basin alignment and therefore do not pose a seismic hazard.

9. Design Earthquake. The San Andreas fault, located 8 miles north of the site, is capable of producing a design base earthquake of magnitude 8.5. A rock acceleration of approximately 0.55g can be expected at the site from such an event. A recurrence interval for a major earthquake along the segment of the San Andreas fault nearest the site is estimated to be from 25 to 160 years.

10. Seismic and Flood Risk. The probability of an earthquake and flood storage occurring simultaneously during the lifetime of an embankment depends upon the return periods (frequency of occurrence) of the earthquake and the flood, the duration of the floodwater storage, and the design life of the embankment. Combined risk in this report is defined as the probability of the simultaneous occurrence of an earthquake and flood storage at least once during the lifetime of the embankment. The following equation[1] was used to compute the combined risk for such an event:

$$\text{Combined Risk} = 1 - \quad 1 - \frac{1}{T_j} \quad 1 - \left(1 - \frac{1}{52T_i}\right)^n \quad {}^{K}$$

where:

T_i = Annual return period of an earthquake exceeding magnitude i

T_j = Annual return period of a flood exceeding storage level j

n = Duration of floodwater storage in weeks

K = Design lifetime of a dam.

A design life of 100 years was assumed for the embankment. The duration of floodwater storage was based on information developed by Hydrology and Hydraulics Branch which is less than one day for all floods. In order to present comparative levels of risk, return periods of 25 and 150 years, corresponding to magnitude 6 and 8.5 earthquakes, respectively, were used to compute combined risks. The results are summarized in TABLE C-4.

TABLE C-1

MAJOR FAULTS WITHIN A 100-MILE

RADIUS OF THE STUDY AREA

Fault	Distance from site (miles)	Fault length (miles)	Maximum Credible Earthquake (Magnitude)	Maximum Site Accelerations in Bedrock (g)*
Agua Caliente	32	50	7.25	0.13
Calico-Newberry	56	50	7.25	0.08
Helendale	30	60	7.5	0.17
Laguna-Salada	64	55	7.25	0.07
Lenwood	41	60	7.25	0.10
Malibu Coast	64	100	7.5	0.10
Newport-Inglewood	72	120	7.5	0.10
Palos Verdes	88	65	7.0	0.04
San Andreas	8	700	8.5	0.55
San Gabriel	79	70	7.5	0.07
San Jacinto	16	170	7.5	0.27
Sierra Madre-Cucamonga	90	75	7.0	0.02
Whittier-Elsinore	40	140	7.5	0.12

*After Schnabel and Seed, 1972

TABLE C-2

EARTHQUAKE EPICENTERS WITHIN

100-MILE RADIUS OF THE SITE

WITH MAGNITUDES 6.0 OR GREATER

Date	Longitude (deg)	Latitude (deg)	Distance* (miles)	Magnitude
21 Apr. 1918	116.9	33.7	29.0	IX-X**
11 Mar. 1933	118.0	33.6	88.7	6.3
25 Mar. 1937	116.3	33.4	24.2	6.0
19 May 1940	115.5	32.7	87.4	6.7
21 Oct. 1942	116.0	33.0	58.3	6.5
10 Apr. 1947	115.6	35.0	86.9	6.2
4 Dec. 1948	116.4	34.0	14.3	6.5
19 Mar. 1954	116.2	33.3	34.0	6.2
9 Apr. 1968	116.1	33.2	41.2	6.4

*From Project Site

**Modified Mercalli Scale Intensity at epicenter

TABLE C-3

EARTHQUAKE EPICENTERS WITHIN

25-MILE RADIUS OF THE SITE

WITH MAGNITUDES 5.0 OR GREATER

Date	Longitude (deg)	Latitude (deg)	Distance* (miles)	Magnitude
25 Mar. 1937	116.3	33.4	24.2	6.0
18 May 1940	116.3	34.1	23.9	5.2
18 May 1940	116.3	34.1	23.9	5.0
12 Jun. 1944	116.7	34.0	23.9	5.1
12 Jun. 1944	116.7	34.0	24.5	5.3
24 Jul. 1947	116.5	34.0	20.2	5.5
25 Jul. 1947	116.5	34.0	20.2	5.0
25 Jul. 1947	116.5	34.0	20.2	5.2
26 Jul. 1947	116.5	34.0	20.2	5.1
4 Dec. 1948	116.4	33.9	14.3	6.5

*From Project Site

TABLE C-4

COMBINED RISK OF SIMULTANEOUS OCCURRENCE

OF FLOOD STORAGE AND EARTHQUAKES

Flood return period, T_j (years)	Duration of storage, n (weeks)	Earthquake return period, T_i (years)	Assumed combined risk per 100 years
10	0.024	25	1.83×10^{-4}
50	0.036	25	5.50×10^{-5}
100	0.048	25	3.66×10^{-5}
200	0.060	25	2.29×10^{-5}
500	0.071	25	1.10×10^{-5}
10	0.024	150	3.05×10^{-5}
50	0.036	150	9.16×10^{-5}
100	0.048	150	6.11×10^{-6}
200	0.060	150	3.82×10^{-6}
500	0.071	150	1.83×10^{-6}

AD-A150 305 WEST MAGNESIA CANYON CHANNEL CITY OF RANCHO MIRAGE RIVERSIDE COUNTY CALIF..(U) ARMY ENGINEER DISTRICT LOS ANGELES CA DEC 83 2/3

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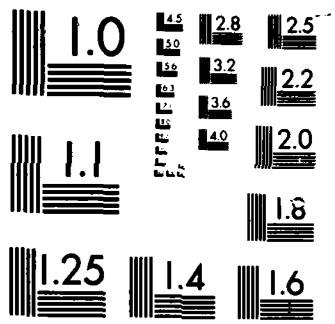


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

FIELD INVESTIGATIONS

11. General. Field investigations were conducted from September through December 1980 and in June 1981 at the site of the debris basin, along the alignment of the channel and at potential borrow areas to determine the design and cost data for constructing the proposed structures. The field investigations consisted of geological reconnaissance and mapping, refractive seismic surveys, auger and diamond-core drilling with water pressure testing, excavating trenches and pits with a Gradall and backhoe, and conducting in-place density and permeability tests.

12. Foundation and Borrow Areas. Geologic mapping of the proposed embankment foundation and borrow areas was conducted using 1:2400 aerial photographs in conjunction with site reconnaissance and investigations. The subsurface investigations at the proposed foundation and upstream borrow areas consisted of excavating 17 test trenches and test pits with a G-660 Gradall and a Case backhoe, drilling 2 NW size diamond-core holes in bedrock with a model B-53 Mobil drill, drilling 1 auger hole to bedrock, and conducting 8 refractive seismic surveys. The trenches were excavated to a maximum depth of 20 feet, were visually examined and disturbed samples of representative materials were obtained for laboratory testing. Seventeen in-situ density tests were conducted in the excavated trenches by the sand cone method, ASTM D 1556, along with 9 constant head and 5 falling head field permeability tests. The locations of the drill holes, geologic test trenches and refractive seismic survey lines are shown on plate C-5. The locations of the test trenches and test pits in the foundation and borrow areas are shown on plate C-10.

13. Channel Alignment. The subsurface investigations along the alignment of the proposed channel improvements consisted of excavating 10 test trenches to a maximum depth of 15 feet with a Gradall. The trenches were visually examined and disturbed samples of representative materials were obtained for laboratory testing. Four in-situ density tests were conducted in the trenches by the sand cone method. The locations of the test trenches along the alignment of the channel are shown on plates C-13 and C-14.

FIELD TESTS AND RESULTS

14. Density Tests. The 21 in-situ sand cone density tests, ASTM D 1556, are in general accordance with EM 1110-2-1907, "Soil Sampling", dated 31 March 1972. A summary of the results of density tests in the foundation and borrow materials, showing dry density versus depth, is presented on plate C-15.

15. Permeability Tests. Permeability tests were conducted on insitu foundation and borrow materials at depths ranging from 5 to 15 feet. The procedure used was in accordance with the requirements of Test Method E-18 of the Earth Manual.[4] A 5-inch inside diameter pipe was pushed into the soil approximately 6 inches and filled with water until the materials immediately beneath the pipe were judged to be saturated. Constant head tests were conducted by maintaining the water level inside the pipe at approximately a 2 foot height for 3 to 5 minutes and recording the weight of water used to maintain that level. Falling head tests were conducted by recording the time required for the water level inside the pipe to drop from one height to another. Constant head tests indicated permeability values (K), ranging from

21 to 77 feet per day. The results of the falling head tests on the same types of materials were more variable, indicating permeability values (K), ranging from 31 to 223 feet per day.

16. Refractive Seismic Surveys. Refractive seismic surveys were conducted at the eight locationss shown on plate C-5 to determine the depth to bedrock and subsurface P-wave velocities along the proposed embankment alignment and in the reservoir area. A twelve-channel signal enhancement seismograph was used, with both explosive charges and a sledge hammer to generate P-waves. The seismic survey lines ranged in length from 99 to 330 feet. Time-distance curves and profiles obtained from the interpretation of the data from these surveys are shown on plates C-6 and C-7.

LABORATORY TESTS AND RESULTS

17. Test Methods.

a. District Laboratory. Grain size analysis tests were conducted at the Los Angeles District laboratory in accordance with EM 1110-2-1906, "Laboratory Soils Testing", dated 30 November 1970. The soils were classified in accordance with the Unified Soil Classification System. Compaction studies and relative density tests were conducted on representative samples of potential embankment and foundation materials in accordance with ASTM test methods D 698 and D 2049.

b. Division Laboratory. Grain size analysis, specific gravity, consolidated undrained triaxial compression with pore pressure measurements, consolidation and permeability tests were conducted on selected samples of

remolded embankment and foundation materials at the South Pacific Division laboratory in accordance with EM 1110-2-1906.

18. Test Results. The results of grain size analysis and classification tests on the excavated materials are shown in the soil logs of test pits and test trenches on plates C-11 through C-14. Summaries of grain size analysis tests, showing upper and lower limits, upper and lower quartiles and mean values, are shown on plate C-15 for materials excavated from the embankment foundation, borrow and channel foundation areas. Also summarized on plate C-15 are results of compaction studies, relative density, drained-consolidated shear strength, undrained-consolidated shear strength and permeability tests. The Mohr's circles representing drained-consolidated shear strengths were derived from pore pressure measurements on the undrained-consolidated triaxial compression tests. Pertinent data on the size of samples, gradations, moisture contents, molding densities, pressures, etc. are shown in Attachment C-1.

ANALYSIS OF DATA

19. Embankment Foundation. The proposed embankment alignment is located across the mouth of Magnesia Spring Canyon where the canyon is approximately 550 feet wide. Interpretation of P-wave velocity data from refractive seismic surveys near the proposed alinement, lines 80-1 through 80-6, indicate that the alluvium in the canyon bottom is composed of at least three layers. The first is a thin, low velocity surface layer representing unconsolidated recent deposits. An intermediate layer is indicated by slightly higher P-wave velocities, 1,400 to 2,200 fps, at depths of 5 to 20 feet. A third layer is

indicated by two higher ranges of P-wave velocities, 2,750 to 4,100 fps at depths from 12 to 20 feet, and 4,600 to 4,900 fps at depths from 16 to 40 feet. The P-wave velocities recorded in the third layer are indicative of moderately consolidated valley fill alluvium, older fan deposits or saturated alluvium. However, groundwater was not encountered during trenching or drilling. Moderately weathered metamorphic basement rocks, at computed depths of 105 to 120 feet, are indicated by P-wave velocities averaging between 12,000 and 15,000 fps measured in the bottom layer. Bedrock was encountered at a depth of 45.8 feet in drill hole D-2, located approximately 40 feet from rock outcrops near the left abutment. A geologic profile along the proposed embankment section is shown on plate C-9.

Data from test pits 80-1 through 80-7 and test trench 80-6, excavated near the proposed alignment of the embankment, indicate that the foundation materials are predominantly non-plastic coarse sands, sandy gravels and silty sands with generally less than 10 percent cobbles and boulders up to 20 inches in diameter and less than 6 percent by weight passing the No. 200 sieve. The in-situ dry densities of 15 tests in the foundation materials, as determined by the standard sand displacement method, ranged from 86.4 to 132.3 pcf as shown on plate C-15. Discounting the one low test, where a test procedure is likely in error, and the two high tests, which were taken in terrace deposits near the right abutment, the densities of the remaining 12 tests ranged from 100.7 to 111.2 pcf with an average value of 106.6 pcf. This corresponds to a range of 87.6 to 96.7 percent of maximum density, as determined by ASTM test method D 698, with an average value of 92.7 percent. If the average maximum density of the vibrated tests (ASTM D 2049) is used as the base of comparison, the in-situ dry densities ranged from 78.8 to 87.0 percent of maximum density,

with an average value of 83.4 percent. The corresponding relative density values indicated by these tests are meaningless since the test procedure used to establish minimum densities was apparently in error and produced an average minimum density of 108.0 pcf. For the depth to which in-situ density tests were taken, no significant change in density with depth is indicated. The permeability values indicated by field tests ranged from 21 to 223 fpd with an average value of 76 fpd.

20. Left Abutment. The left abutment is composed of gently dipping metasediments and small intrusive granitic bodies of the Palm Canyon Complex. The bedding of the metasediments dip generally downstream to the northeast and into the abutment. Three test trenches were excavated with a backhoe at the toe of the left abutment to trace the bedrock surface below the alluvium. These trenches, along with seismic surveys and drilling investigations, indicate that the bedrock surface slopes steeply downward beneath the channel alluvium. The degree of surface weathering varies, depending on the composition of the individual rock layers. Harder quartz veins and marble strata are only slightly weathered whereas thin schistose zones are moderately weathered. Treatment of foundation rock in the abutment would be minimal since there would be no long term water retention behind the debris basin. Slope wash colluvium and residual soil are present on the surface and are generally less than one foot thick on the left abutment. Some scaling to slope back near-vertical outcrops and dental excavation of the more weathered rock would be required at the embankment-abutment contact. The surface rocks are not extensively jointed or fractured. Although core recovery in hole D-3, which was drilled through representative rock immediately downstream of the centerline of the left abutment, was only 88.2

percent and the Rock Quality Designation was 22 percent (see legend on plate C-8), pressure test data indicate a low rock mass permeability. The only water take in D-3 was 1.0 to 1.4 gpm at 10 psi in the 8.1 to 15.6 feet interval which indicates a mass permeability of 0.6 fpd. The log of D-3 is shown on plate C-8. Location of test trenches at the left abutment are shown on plate C-5 and profiles of the trenches are shown on plate C-9. This abutment would be unsuitable for an unlined, detached spillway site because of the unfavorable topography and the large amount of required excavation.

21. Right Abutment. The hills forming the area immediately above the right abutment are composed of rock from the same Palm Canyon Complex present on the left abutment of the debris basin. The attitude of the bedding and foliation strikes to the northwest and dips downstream and into the abutment. Bedrock does not outcrop at the elevation of the top of the proposed embankment, but is covered by older terrace deposits. The terrace deposits are composed of sands, gravel and cobbles which are loose at the surface even though the slopes are near vertical in the main channel. Refractive seismic survey lines 80-7 and 80-8, conducted on the terrace, show seismic P-wave velocities ranging from 1,050 to 2,300 fps to a depth of at least 40 feet and indicate a steeply dipping bedrock surface at the south end of line 80-7.

Test Trench 80-D was excavated by backhoe near the upstream portion of the right abutment (see pl. C-5) to establish the depth of bedrock beneath the rock-alluvium contact at the toe of the slope and to visually inspect the exposed rock. The cross-section of the trench is shown on plate C-9. Excavation was along the small, relatively blunt nose forming the northern most extension of the rock outcrop in this area. Because of the near vertical

slope and the south-southwest strike of the bedrock into the hill on both sides of the blunt tip, the contact could not be followed away from the channel. The rock in the near vertical face of the northern most exposure continues to a depth of 20 feet (elev. 504 feet) below the ground surface where it forms a nearly level bench which extends outward from the vertical face approximately 10 feet. At this point the bedrock surface resumes its downward plunge at an angle of approximately 45° toward D-1, which it intersects at 22.1 feet (elev. 477 feet) below the ground surface. An in-place knob, with a maximum diameter of 6 feet, is attached to the bench. The subsurface bedrock exposed in the trench was moderately weathered and massive, exhibiting no major jointing along the face.

Core recovered from D-1, drilled near the right abutment, was highly fractured, indicating numerous broken zones. However, the water infiltration rate during pressure testing was very low. Flow rates of 0.5 to 1.5 gpm at 5 and 20 psi, respectively, indicate mass permeability values for D-1 ranging from 0.1 to 0.2 fpd. Core loss in D-1 ranged from 0 to 79 percent for individual runs. The log of D-1 and pressure test data are shown on plate C-8. The terrace materials upstream and downstream of the abutment would be highly erodible to spillway flows should a detached spillway be considered on this side of Magnesia Spring Canyon.

22. <u>Embankment Materials</u>. Based on field observations and results of classification, compaction and relative density tests conducted on materials representative of potential borrow areas, only one general type of material is available near the site for use in construction.

Materials suitable for use as an impervious core are not available near the site. The closest location known to have relatively impervious materials in sufficient quantities is at the Mecca Hills near the Salton Sea, approximately 25 miles from Rancho Mirage. These materials, previously tested at the Division laboratory as part of the investigation for La Quinta Dam, are highly plastic sandy clays with permeabilities of less than 0.1 fpd.

The embankment material would be predominantly non-plastic, noncohesive, coarse sand with varying amounts of gravel and generally less than 6 percent by weight passing the No. 200 sieve. Results of tests on materials from the borrow area near the proposed embankment indicate the following range of properties:

Maximum dry density:	d = 111-127 pcf (ASTM D 698)
	d = 124-132 pcf (ASTM D 2049)
Optimum moisture content:	w = 6-10 percent
Undrained shear strength:	ϕ = 21-34^{o}
	c = 0
Drained shear strength:	ϕ' = 37-38^{o}
	c' = 0
Permeability:	K = 21-223 fpd

23. Channel Foundation. Data from test trenches 80-7 through 80-16, excavated along the existing alignment of the channel, indicate that the channel foundation materials are predominantly non-plastic sands, gravelly sands and silty sands with generally less than 10 percent cobbles up to 12 inches in the least dimension and less than 10 percent by weight passing the No. 200 sieve. Occasional layers of fine-grained, non-plastic silty sands and

sandy silts were encountered, mainly at the downstream end of the channel. Larger quantities of cobbles and boulders were encountered in test trenches 11 and 12. Bedrock outcrops at Stations 35+00 to 36+50, 55+50, and 61+50 intrude into the existing channel alignment from the hills on the northwest side of the channel. The in-situ dry densities of 4 tests in the foundation materials, as determined by the standard sand displacement method, were all between 111 and 112 pcf. This corresponds to approximately 97 percent of maximum density as determined by ASTM test method D 698, and 87 percent of maximum density as determined by ASTM test method D 2049.

EMBANKMENT DESIGN

24. Design Values. The selected design values are based on the results of detailed laboratory testing conducted on disturbed samples of representative foundation and borrow materials compacted to 95 percent of maximum density (ASTM D 698). The 95 percent value was chosen to approximate the insitu density of the foundation materials and as a conservative assumption of the expected densities in the compacted embankment materials. The samples selected for testing are generally representative of the narrow range of gradations between the upper and lower quartiles shown on plate C-15. The moisture-density relationships established by compaction studies and in-situ foundation tests were used to determine the dry and drained unit weights. The saturated unit weight was determined by calculating the volume of voids at 95 percent of maximum density and assuming these voids were filled with water. The shear strengths selected are interpretations of the triaxial compression test data following the guidelines outlined in paragraph 9b of EM 1110-2-1902, "Stability of Earth and Rock-Fill Dams," dated 1 April 1970. The coefficient

of permeability is a conservative selection based on tests of materials compacted to 95 percent of maximum density. The selected design values for the foundation and embankment materials, at 95 percent of maximum density, are shown in TABLE C-5.

TABLE C-5

EMBANKMENT AND FOUNDATION DESIGN VALUES

Material	Shear Strength "R" C (PSF)	Shear Strength "R" Ø (DEG)	Shear Strength "S" C' (PSF)	Shear Strength "S" Ø' (DEG)	Permeability K (FPD)	Unit Weight m* (PCF)	Unit Weight sat* (PCF)
Embankment	0	30	0	36	30	120	140
Foundation	0	28	0	34	70	105	130

*Based on 95 percent maximum density (D698)

25. Embankment Section. The embankment has a homogeneous section constructed with materials obtained from the required excavation of the basin and channel and a 20-foot wide crest with 1V on 3H upstream and downstream slopes. A zoned dam at this location is impractical because of the high cost of importing impervious core material, therefore, a 4-inch thick reinforced concrete slab on the upstream face would extend from the crest of the embankment to a depth of 10 feet below the excavated toe. It would reduce seepage through the embankment from impounded water, prevent erosion of the embankment materials due to floodflows and off-road vehicles, and allow accumulated debris in the basin to be excavated without removing embankment materials. A downstream drainage blanket would be provided to prevent piping and erosion by collecting through and underseepage. A rock toe section would

be provided at the downstream toe of the embankment to protect the exit of the blanket drain and to prevent access by off-road vehicles. A typical cross section of the proposed embankment is shown on plate C-16.

26. <u>Foundation Treatment</u>. After clearing and grubbing, the embankment foundation would be excavated to a depth of 10 feet and proof rolled with a vibratory roller. The foundation excavation is required to remove loose materials, prevent excessive settlement and to increase the shear strength and lower the permeability at the base of the embankment. The materials obtained from the foundation excavation would be stockpiled and used in the embankment fill.

The residual soils at the embankment contact area on the left abutment would be removed to bedrock and the bedrock surface cleaned. Some scaling and dental excavation would be required to slope back rock outcrops so that fill materials could be placed and compacted next to them.

The near-vertical face of the alluvial terrace that forms the right abutment would be sloped and stepped back at a 1V on 2H slope to produce a suitable surface against which to place and compact embankment fill materials. The materials obtained from the abutment excavation would be stockpiled and used in the embankment or as landscape fill material, if needed.

27. <u>Outlet Works</u>. The intake tower and outlet pipe would be founded on alluvium and compacted fill. The foundations would be proof rolled with a vibratory roller as necessary to prevent excessive settlement and any potential failure of the outlet works structures.

28. Slope Stability. A computer program for slope stability analysis, using the circular arc method of slices, was used to determine the locations of critical failure surfaces for the design conditions listed in Table 1 of EM 1110-2-1902. Design assumptions and results of stability calculations are presented for each condition in the following paragraphs.

a. End of Construction. The embankment and foundation materials were analyzed using consolidated-drained strengths since it is improbable that excessive pore pressure would develop in the embankment or foundation during construction because of the granular materials, the depth to groundwater and the limited range of moisture contents to be specified for placement. Minimum safety factors for deep-seated arcs were significantly higher than for the shallow arcs. The minimum safety factors calculated for the embankment are listed below and the corresponding sliding surfaces are shown on plate C-17.

END OF CONSTRUCTION

MINIMUM SAFETY FACTORS

Corps Minimum	Upstream Slope	Downstream Slope
1.3	2.2	2.2

b. Steady Seepage and Partial Pool. The embankment was analyzed using consolidated-undrained strengths for the embankment and foundation materials. The downstream slope was analyzed for the steady seepage condition with the water surface at spillway crest and the phreatic surface conservatively extending from spillway crest elevation at the upstream face to the downstream toe. The effect of the gravel drain and concrete facing on the

phreatic surface was not considered. The upstream slope was analyzed for the partial pool condition with the phreatic surface as described above. Since the structure would have an ungated outlet and a maximum pool detention time of less than one day, retention of significant amounts of water for extended periods of time would occur only if blockage of the outlet should occur during flood operations. Therefore these two conditions are considered unlikely to occur and the analysis is very conservative. The minimum safety factors calculated for the embankment are listed below and the corresponding sliding surfaces are shown on plate C-17.

STEADY SEEPAGE AND PARTIAL POOL

MINIMUM SAFETY FACTORS

Corps Minimum	Upstream Slope	Downstream Slope
1.5	1.7	1.8

c. Drawdown. The upstream slope of the embankment was analyzed for the conditions of drawdown from the maximum water surface and spillway crest elevations to the invert of the intake structure. Consolidated-undrained strengths were used below the phreatic surface for the embankment and foundation materials with the phreatic surface extending from the water surface at the upstream face to the downstream toe. Consolidated-drained strengths were used above the phreatic surface. The phreatic surfaces used in the analysis are conservative because the expected period of pool storage is short, less than one day. The minimum safety factors calculated for each condition are listed below and the corresponding sliding surfaces are shown on plate C-17.

DRAWDOWN

MINIMUM SAFETY FACTORS

Maximum Water Surface Corps Minimum	Maximum Water Surface Calculated	Spillway Crest Water Surface Corps Minimum	Spillway Crest Water Surface Calculated
1.0	1.7	1.2	1.7

d. Earthquake. Rancho Mirage is located in seismic risk zone 4 with a designated seismic coefficient of 0.15g. This factor was used in a pseudo-static computer analysis of the embankment using the circular arc method of slices to determine the locations of critical failure surfaces. The embankment was evaluated for the end of construction and steady seepage conditions with the added horizontal seismic driving force to determine the minimum factors of safety under seismic loading conditions. The minimum safety factors calculated for each condition are listed below and the corresponding sliding surfaces are shown on plate C-17.

EARTHQUAKE LOADING

MINIMUM SAFETY FACTORS

Corps Minimum	End of Construction Upstream Slope	End of Construction Downstream Slope	Steady Seepage Downstream Slope
1.0	1.4	1.4	1.0

29. <u>Seismic Induced Slope Displacement</u>. In addition to the pseudo static seismic analysis, earthquake induced permanent slope displacements for the calculated yield accelerations were estimated using the Newmark-Ambreyseys-Sarma procedure. Estimating the amount of earthquake induced permanent slope displacement allows for the affect the displacement would have on the freeboard of the embankment. Since the yield acceleration is based on a loading condition which produces a factor of safety equal to 1.0 for an assumed critical failure plane, the analysis implies the possibility of an earthquake occurring on a local or regional fault having a great enough magnitude to produce the yield acceleration. The design earthquake was determined to be an 8.5 magnitude event on the San Andreas Fault, 8 miles from the site, producing a rock acceleration of approximately 0.55g beneath the embankment. Ground accelerations at the base of the embankment were assumed to be the same as the bedrock accelerations due to the shallow depth to bedrock and the relatively narrow width of the canyon. A shear wave velocity of 1500 fps was assumed for the compacted embankment material.

The yield accelerations were determined for different embankment heights using a circular arc pseudo-static slope stability analysis. The yield acceleration is the seismic coefficient which produces a factor of safety equal to 1.0 for an assumed failure plane. Yield accelerations for arcs in an embankment with 1V on 3H slopes are shown on plate C-17. The peak seismic embankment motions were determined by amplifying the peak ground motions. The amplification factors were determined from curves developed by Sarma and Ambreyseys(2) for embankments on a rigid base. The amplification factors of ground surface motions through an embankment founded on a stiff alluvial base would not vary significantly from the amplification factors for an embankment

founded on a rigid base. Accelerations and velocities were determined for each height of the embankment multiplying the ground motion by the amplification factor.

The ratio between the yield acceleration and seismic acceleration of the embankment (Ay/Aeq) was determined at various embankment heights. Standardized maximum displacements, Us, versus Ay/Aeq were determined from a Newmark analysis[3] for the design earthquake. The standardized displacements were determined for earthquake motions scaled to a peak acceleration = 0.5g and a peak velocity = 30 in/sec.

The estimated permanent displacement, Um, is calculated at each height in the embankment from the following relationship.

$$Um = \frac{Us \quad V^2}{1800 A_{EQ}}$$

where:

Us = standardized maximum displacement

= factor for critical surface = cos $\frac{(\phi - B)}{\cos \phi}$

V = amplified peak ground velocity

A_{EQ} = amplified peak ground acceleration

B = direction of resultant shear force

ϕ = average friction angle

The assumed failure surfaces for each height in the embankment and the maximum displacements estimated by this procedure for various embankment side slopes are shown on plate C-17. Also shown are the estimated displacements for a set of shallow arcs in an embankment with 1V on 3H slopes. The estimated displacements would not significantly affect the stability of the embankment since it has 5 feet of freeboard and the maximum estimated displacement for the recommended embankment slope is less than 1 foot.

30. Liquefaction Potential. The liquefication potential of the foundation materials when subjected to seismic loading conditions and the resulting stability of the proposed embankment were evaluated. The conditions analyzed were foundation saturation, in-situ densities, grain size distribution.

a. Foundation Saturation. Drill hole and refractive seismic data indicate that groundwater normally exists only at the bedrock/alluvium contact, which, at the location of the proposed embankment, is about 100 feet deep. Because of the coarse nature and high permeability of the alluvial materials, saturation of large portions of the foundation would occur only during periods of floodwater impoundments. The duration of standard project flood impoundments would be less than one day. Therefore, the probability of a major earthquake occurring during a saturated foundation condition is very low. The range of combined risk, presented in TABLE C-4 is from 0.000183 for a flood return period of only 10 years and an earthquake return period of 25 years to 0.00000183 for a flood return period of 500 years and an earthquake return period of 150 years.

b. Density. In-situ density tests and laboratory compaction tests indicate that the foundation materials have densities ranging from 87 to 97 percent of maximum density (ASTM D 698), with an average value of 93 percent. These materials, based on density alone, would be moderately

resistant to liquefaction. The top 10 feet of the foundation materials under the embankment will be removed and replaced with material compacted to higher densities, improving its resistance to liquefaction.

c. Grain Size. The results of grain size analysis and classification tests indicate that the foundation materials are predominantly well graded, non-plastic, noncohesive gravelly sands. The percent of material larger than the No. 4 sieve ranges from 1 to 59 percent with a mean value of 25 percent. The percent of material smaller than the No. 200 sieve ranges from 1 to 17 percent with a mean value of 4 percent. The uniformity coefficient of the mean gradation is 10, indicating a reasonably well-graded sand. The D_{20} size of the mean gradation is 0.45 mm, however the D_{20} size of the fine gradation limit is only 0.08 mm, which is associated with moderate resistance to liquefaction.

d. Stability Analysis. The stability of the embankment was analyzed under the worst combination of conditions assuming that a major earthquake occurs during a storm of sufficient length and intensity that the foundation is completely saturated, with water flowing on the surface. This condition is highly conservative since the debris surcharge upstream is ignored and surface water downstream where the alluvial fan spreads out is extremely unlikely. Nevertheless, buoyant unit weights and undrained shear strenghts for the foundation materials and the embankment materials below the phreatic surface were used in a computer analysis of slope stability, using the circular arc method of slices to determine the location of critical failure surfaces and minimum factors of safety. In addition, the foundation materials upstream and downstream of the embankment toes, areas not receiving foundation treatment with a vibratory roller, were assumed to have liquefied to a depth of 50 feet and have no shear strength. Minimum factors of safety for failure arcs

passing through the foundation and embankment, with a horizontal seismic force of 0.15g, were calculated to be 1.1 for both the upstream and downstream slopes.

e. <u>Conclusions</u>. Although the simultaneous occurrence of events necessary to produce the required conditions is highly unlikely, limited liquefaction of the foundation materials at the site of the proposed embankment would be possible for those conditions. Stability analysis indicates, however, that catastrophic failure of the embankment would not occur. Relatively loose cohesionless sands in the foundation would be more likely to undergo moderate settlement rather than liquefaction since high pore water pressures would be quickly dissipated. Denser cohesionless sands in the embankment and the rolled foundation area would be subject to limited liquefaction with a limited strain potential, if they were saturated. The resultant displacements would be less than the 5 feet of freeboard that is available during a probable maximum flood, and very much less than the 23 feet between the spillway crest and the top of the embankment.

31. <u>Settlement and Subsidence</u>. The settlement and subsidence along the centerline of the proposed embankment due to the loading of the foundation and the consolidation of the embankment materials are expected to be minimal. The foundation treatment requires removal and recompaction of the top 10 feet of the foundation materials. Due to the coarse nature of the foundation and embankment materials, the settlements would occur rapidly with most of the settlements occurring during construction. Large differential settlements are not expected due to the relatively low maximum height of the embankment (34 feet), and the cohesionless materials are not susceptible to cracking.

32. Seepage. A 4-foot thick by 100-foot wide downstream gravel drainage blanket would be provided to prevent piping and erosion by collecting the through and underseepage. Seepage through the embankment could occur if the concrete upstream facing were cracked due to the seismic, settlement or construction activity. Underseepage is expected due to the relatively high permeability and relatively loose condition of the foundation materials below the 10 foot stripping depth. Both conditions are conservative assumptions since detention times for the debris basin will be low and the groundwater surface is not likely to be near the ground surface in the vicinity of the proposed embankment. Through seepage exiting near the downstream toe is especially unlikely since the foundation materials would be more permeable than the compacted embankment materials and since the spillway crest would be only 11 feet higher than the excavated basin. The location of the drainage blanket in the embankment is shown on plate C-16.

CHANNEL DESIGN

33. Design Values. The adopted design values are based on the results of detailed laboratory testing conducted on disturbed samples of representative materials from the channel foundation. Bearing capacity was determined according to the methods outlined in EM 1110-2-1903, "Bearing Capacity of Soils", dated 1 July 1958. Terzaghi's bearing capacity factors were used in the general bearing capacity formula for local shear failure. The allowable

bearing pressure, with a safety factor of 3, was determined to be 5800 psf. Bearing capacity based on settlement was determined according to the following relationship from "Foundation Analysis and Design" by J. E. Bowles.

$$q_a = 1.2\ (N-3)\ \frac{B+1}{2B}^2\ W^1 K_d$$

where:

q_a = allowable net increase in soil pressure over existing soil pressure for a settlement of 1 inch (ksf)

N = standard penetration number

B = width of footing (ft), (1/2 of channel width assumed)

W^1 = water reduction factor = 1

Kd = depth factor = $1+\frac{D}{B}$

Using a standard penetration test blow count of 30 for medium dense sand that has been proof rolled to 95 percent of maximum density, a footing depth of 0 feet, a water reduction factor of 1 and an allowable settlement of 1/4 inch, the allowable bearing pressure was calculated to be 2270 psf. This was the controlling factor in choosing the design bearing capacity shown in TABLE C-6. The equivalent-fluid pressure for the channel backfill material is based on the saturated unit weight of the materials when compacted to 90 percent of maximum density (ASTM D 698) and an angle of internal friction of 28 degrees for typical channel backfill materials in the undrained condition. The moist

and saturated unit weights were determined for backfill materials compacted to 90 percent and foundation materials compacted to 95 percent of maximum density. The selected design values for the channel foundation and backfill materials are shown in TABLE C-6.

TABLE C-6

CHANNEL DESIGN VALUES

Material	Bearing Capacity (PSF)	Equivalent Fluid Pressure (PCF)	Unit Weight m (PCF)	Unit Weight sat (PCF)
Channel Foundation	2000	-	120	140
Backfill Behind Channel Walls	-	48	114	133
Compacted Fill Under Channel	2000	-	120	140

34. Channel Sections. The channel would be a concrete-lined rectangular section, 20 feet wide, typically 8 to 10 feet deep and 1.4 miles long. It would be founded on undisturbed streambed materials. Temporary side slopes for channel excavation would be 1V on 1H. Permanent side slopes would be 1V on 2H.

35. Foundation Treatment. Foundation treatment for the proposed entrenched channel would consist of excavating to grade, removing cobbles and boulders, and proofrolling the invert with a vibratory roller. Proofrolling would be required to minimize settlement of the granular foundation materials during construction. Some rock excavation near the side of the channel would be required at Stations 35+00 to 36+50, 55+55 and 61+50. Blasting may be required.

36. Seepage. Weep holes would be provided at the base of the channel walls to relieve hudrostatic pressure due to local runoff during storms. The walls then would be designed to withstand the lateral pressure of drained backfill material. A subdrain system under the channel invert would not be required due to the granular nature of the foundation materials and the great depth to groundwater.

CONSTRUCTION MATERIALS

37. Embankment Materials. Approximately 160,000 cubic yards of material would be required to construct the proposed debris basin embankment and access roads. Approximately 250,000 cubic yards of suitable material would be available from the required excavation of the basin and channel. Assuming a 10 percent volume loss due to transportation, compaction and removal of oversize cobbles and boulders, about 225,000 cubic yards of material, or approximately 1.4 times the required amount would be available for construction.

38. Sources of Cement. Sufficient Type II, low alkali cement suitable for concrete construction would be available from cement plants at Colton, California about 65 miles north of the project site; or from Mojave, California 175 miles north of the project site.

39. Sources of Pozzolans. Type F pozzolan suitable for concrete construction would be available from plants at St. Johns, Arizona, approximately 505 miles from the project site; or from plants near Page, Arizona approximately 550 miles from the project. Commercial concrete plants in the project area use these sources in their commonly produced concrete mixtures.

40. Sources of Aggregates. Aggregates suitable for portland cement concrete construction would be obtained in ample quantities from commercial aggregate sources near Beaumont, Whitewater and Garnet, California. Information on these sources can be found in indexes 1, 2 and 3 for latitude 33 degrees N and longitude 112 degrees W at Waterways Experiment Station, Technical Memorandum No. 6-370, dated September 1953, titled "Test Data, Concrete Aggregates in Continental United States," Volume 2 Area 3, Western United States. Aggregates from these plants have been used previously in a wide variety of concrete products.

41. Sources of Water. Sufficient water suitable for use in concrete construction would be available from local water districts.

42. Sources of Stone. Quarries within approximately 70 miles of the project site which would be able to produce stone of sufficient size in the quantities required at the time of construction are located near the cities of Riverside, Joshua Tree, Cabazon and Hemet. There are no known dependable sources of riprap within 10 miles of the site.

CONSTRUCTION PROCEDURES

43. Embankment. The debris basin embankment would be constructed with readily available heavy construction equipment. The borrow material would be excavated from the basin area in such a manner that a uniformly blended embankment material would be produced. The moisture content would be specified to be the optimum required for maximum compaction. Cobbles and boulders larger than 9 inches in the minimum dimension would be raked on grade to the downstream edge of the embankment. The materials would be placed in

12-inch uncompacted horizontal lifts and compacted with a vibratory roller to 85 percent of the maximum density determined by ASTM test method D 2049. Relative density would not be used for construction control due to the difficulty of determining the minimum density of the materials. Instead, the maximum vibrated density (ASTM D 2049) would be used as the base of comparison for determining the percent compaction of placed materials. The specified percent of maximum density will be equal to or greater than 80 percent relative density.

44. Access Roads. Access roads to the embankment and basin would be constructed in the same manner as the embankment except that the top 12 inches of fill would be compacted to 95 percent of maximum density (ASTM D 2049).

45. Channel. The channel excavation would be backfilled with material from the required excavation for the channel. The moisture content would be specified to be the optimum required for compaction. Cobbles and boulders larger than 9 inches in the minimum dimension would be removed. The material would be placed in 12-inch uncompacted horizontal lifts and would be compacted to 80 percent of maximum density (ASTM D 2049).

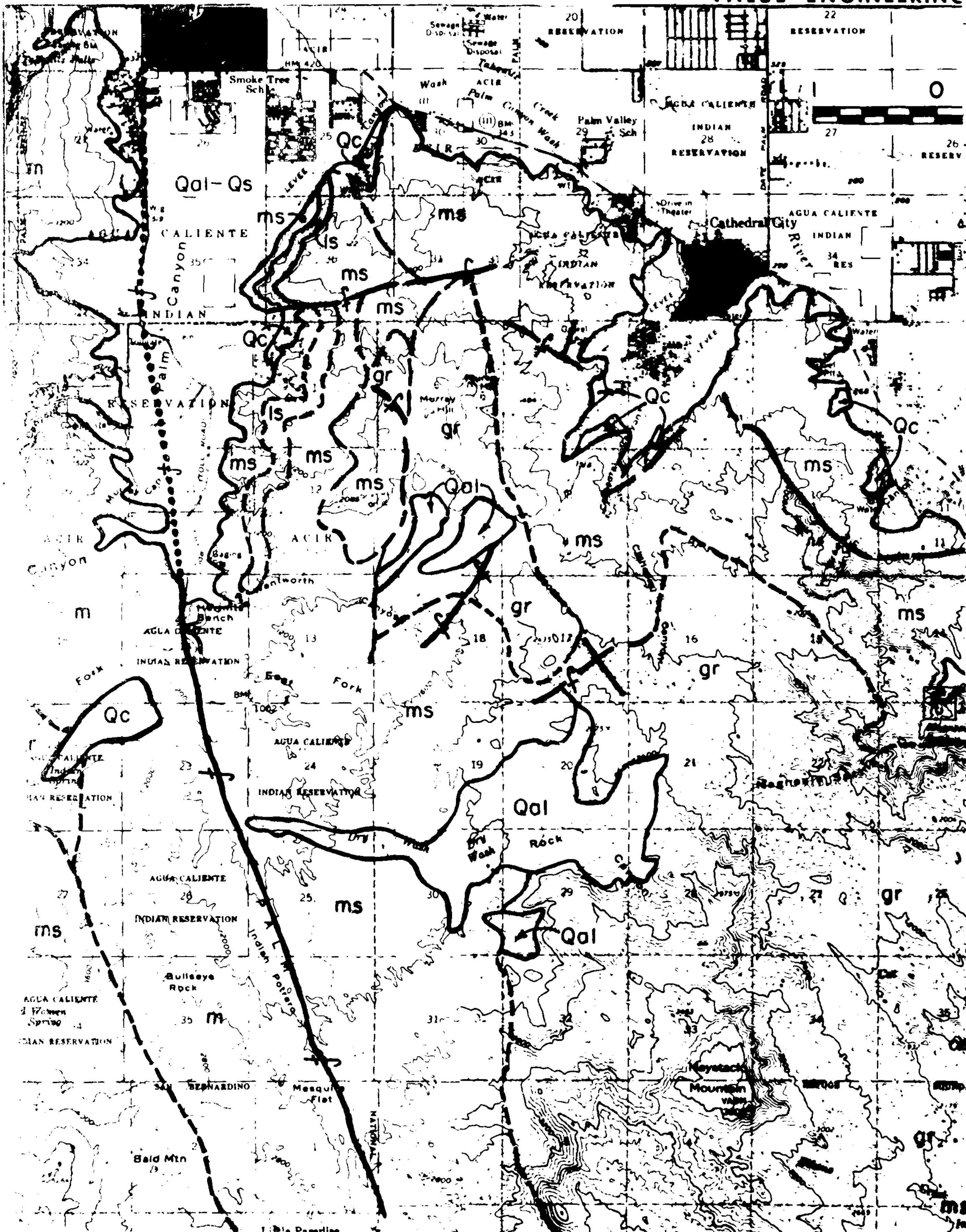
REFERENCES

1. Hynes, M.E., "Notes on Joint Occurrence of Earthquakes and Floods", U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, March 1978.

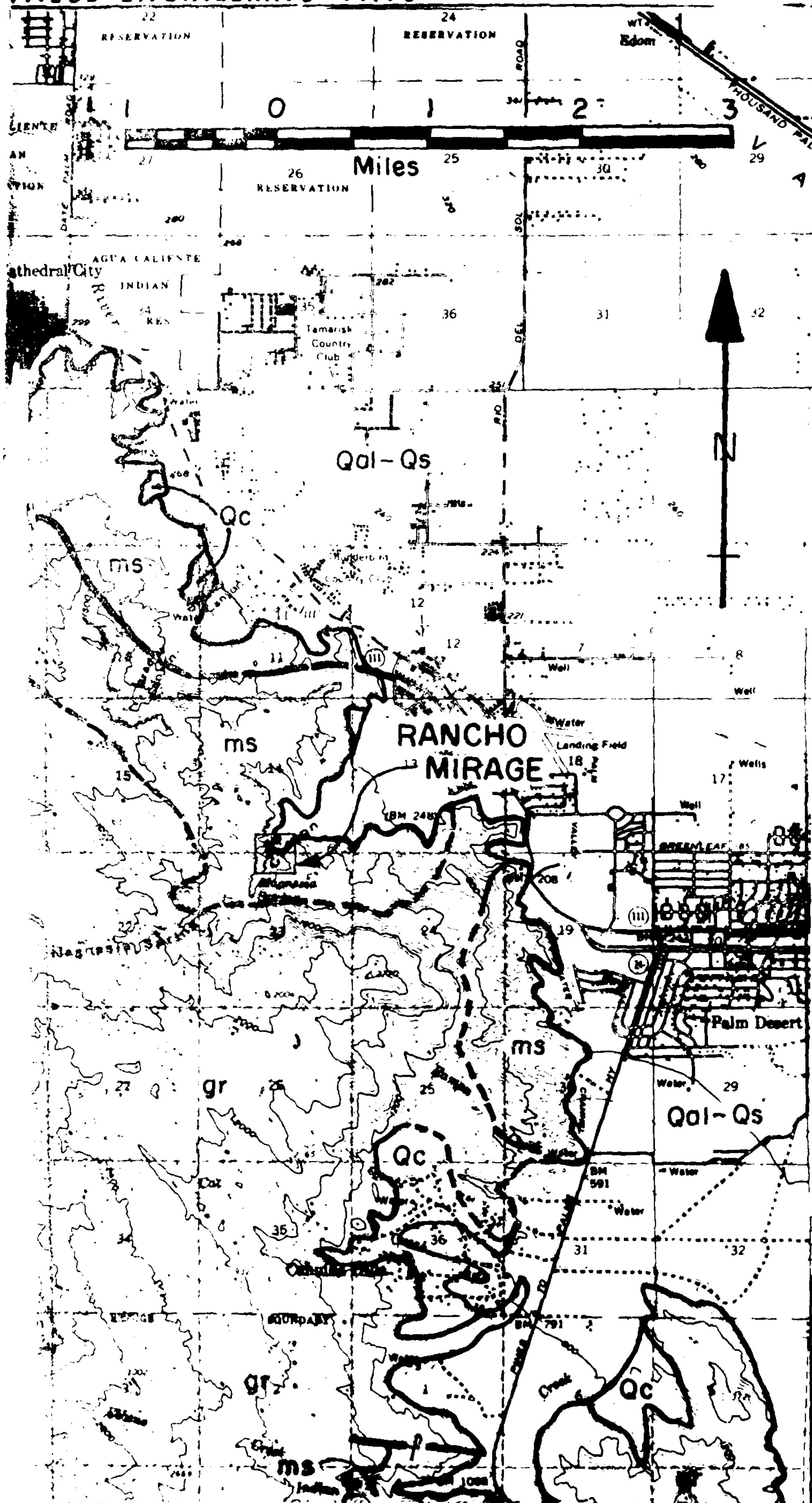
2. Ambreyseys, N.N., and Sarma, S.K., "The Response of Earth Dams to Strong Earthquakes," Geotechnique, Vol. 17, 1967, pp. 181-213.

3. Franklin, A.G., and Chang, F.K., "Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis," Misc. Paper S-71-17, Report 5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1977.

4. U.S. Department of the Interior, Bureau of Reclamation: Earth Manual, A Water Resources Technical Publication, Second Edition, 1974, pp. 575-578.



RESERVATION
Smoke Tree Sch
ACIR
Palm Canyon Wash
Palm Valley Sch
AGUA CALIENTE
INDIAN
RESERVATION
Cathedral City
Drive in Theater
Qal-Qs
Qc
ms
ls
gr
Qal
m
Canyon
Palm
Murray Hill
Wentworth
East Fork
Dry Wash
Rock
Bullseye Rock
Mesquite Flat
SAN BERNARDINO
Bald Mtn
Haystack Mountain



LEGEND

Qs	Recent dune sand
Qal	Recent alluvium
Qt	Quaternary nonmarine terrace deposits
Qc	Pleistocene nonmarine
gr	Mesozoic granitic rocks.
m	Pre-Cretaceous metamorphic rocks
gr-m	Pre-Cenozoic granitic and metamorphic rocks
ls	Pre-Cretaceous metamorphic limestone or dolomite
ms	Pre-Cretaceous metasedimentary rocks
	Approximate formation contact, dotted where concealed, queried where infered
	Approximate location of fault; dotted where concealed, queried where infered. May form contact between formations.

NOTES

1. Modified from Geologic Map of California, Santo Ana Sheet, Division of Mines and Geology, 1965 Scale 1:250,000.

SYMBOL	DESCRIPTIONS	DATE	AP

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WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

REGIONAL GEOLOGY MAP

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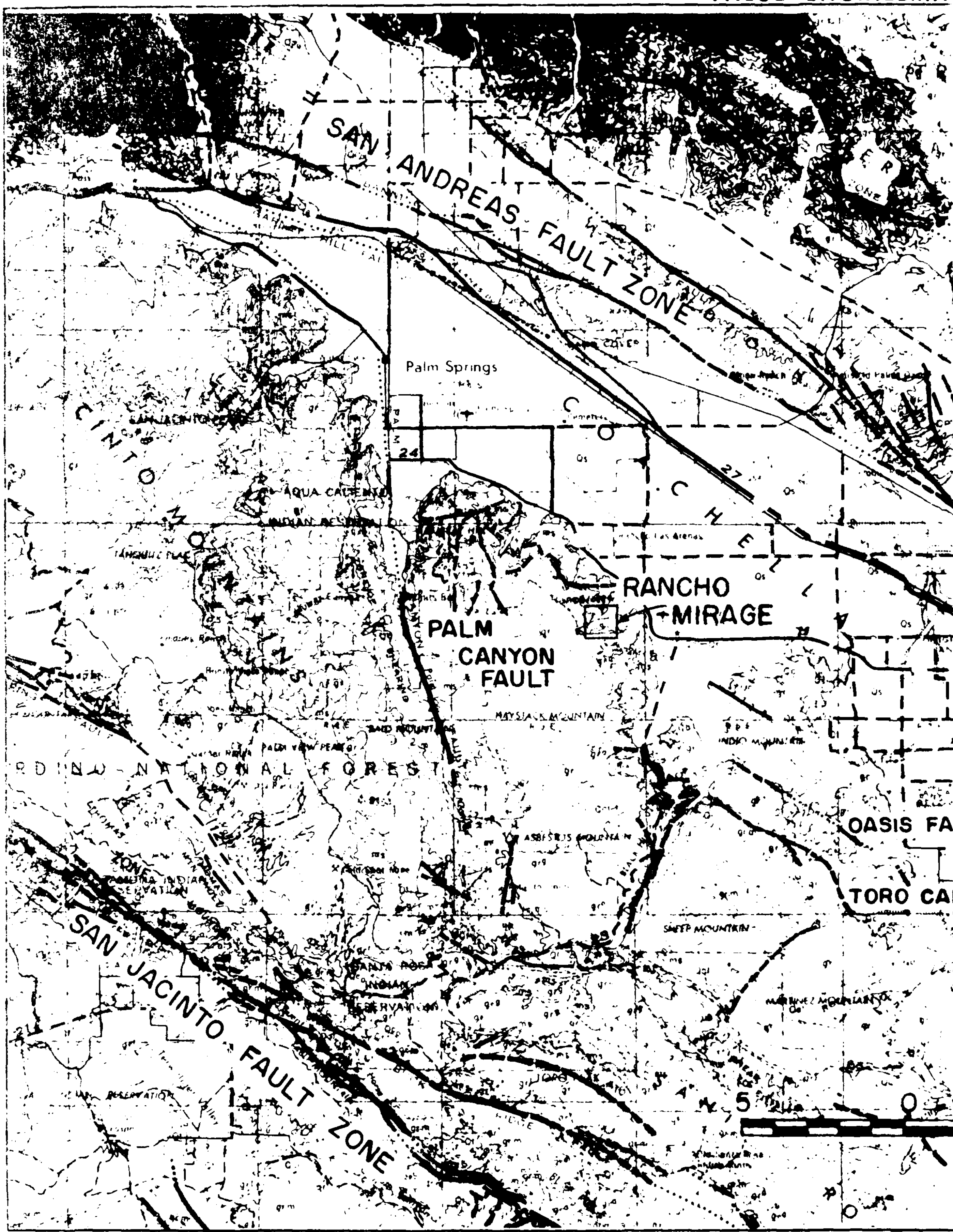
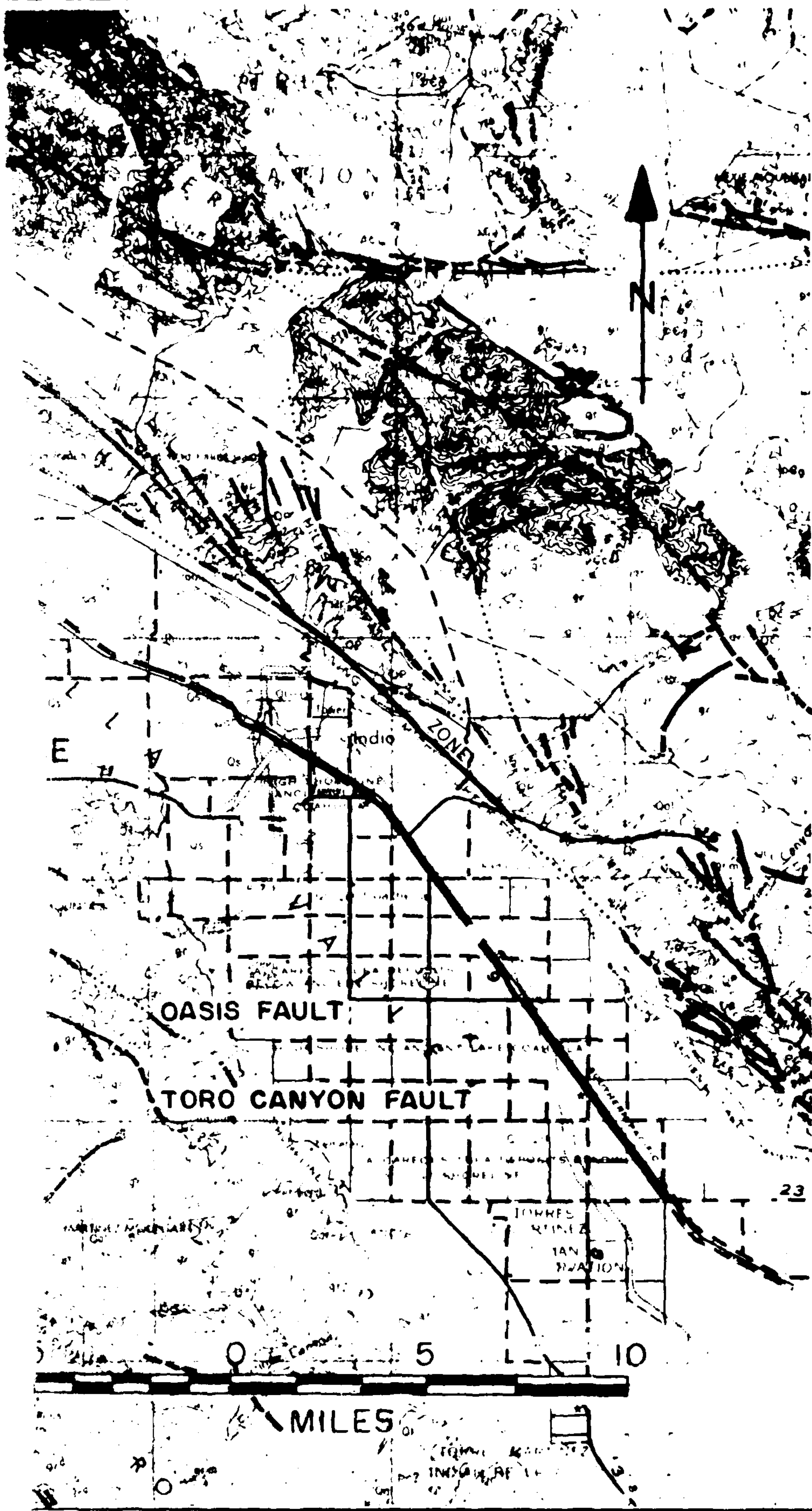
SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO

PLATE C

II


SAN ANDREAS FAULT ZONE
Palm Springs
RANCHO MIRAGE
PALM CANYON FAULT
OASIS FA
TORO CA
SAN JACINTO FAULT ZONE
HAYSTACK MOUNTAIN
SHEEP MOUNTAIN
NATIONAL FOREST
5
0



NOTES
See plate C-1 for legend and notes

SYMBOL	DESCRIPTIONS	DATE

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CHANNEL AND DEBRIS BASIN

REGIONAL FAULT MAP

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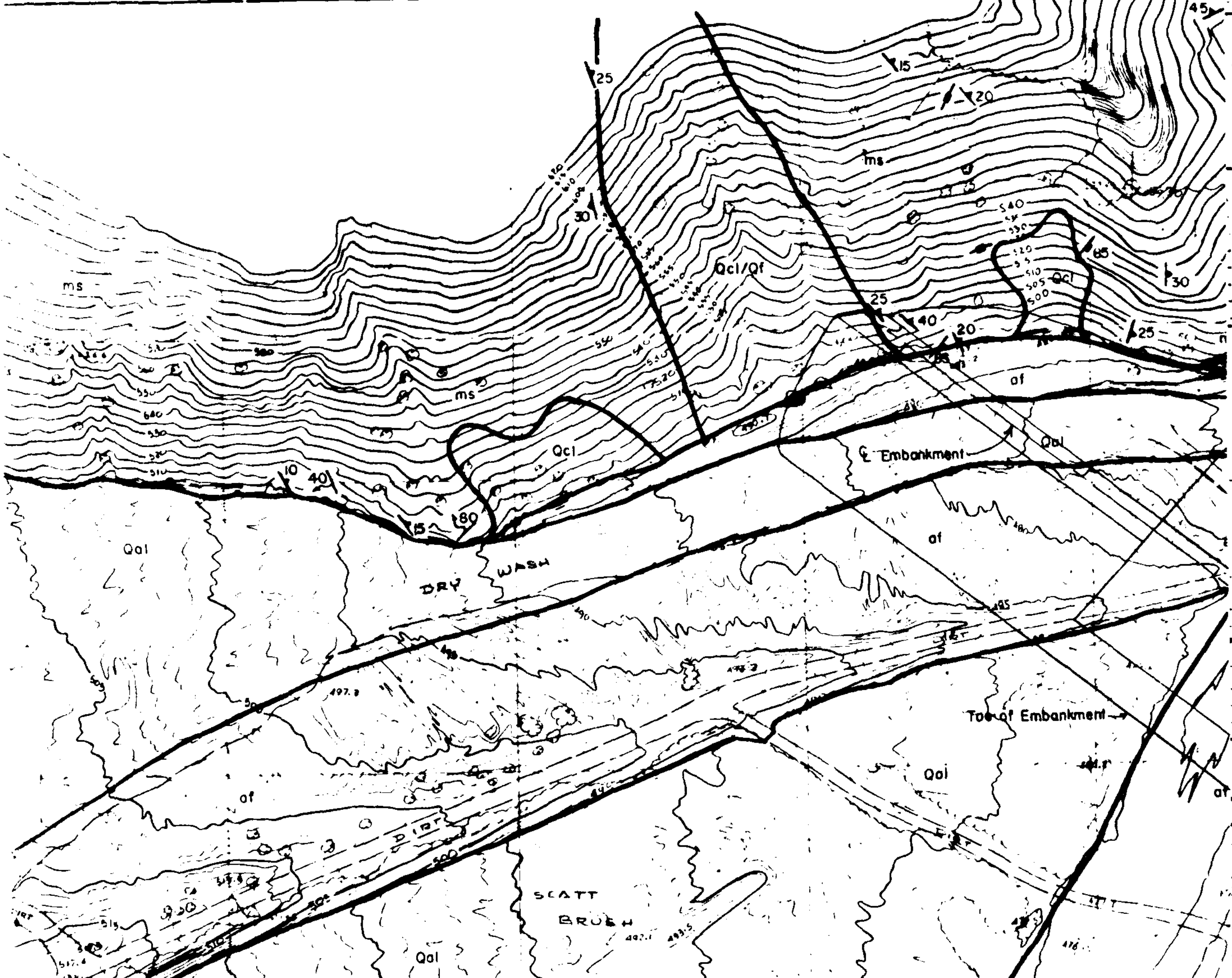
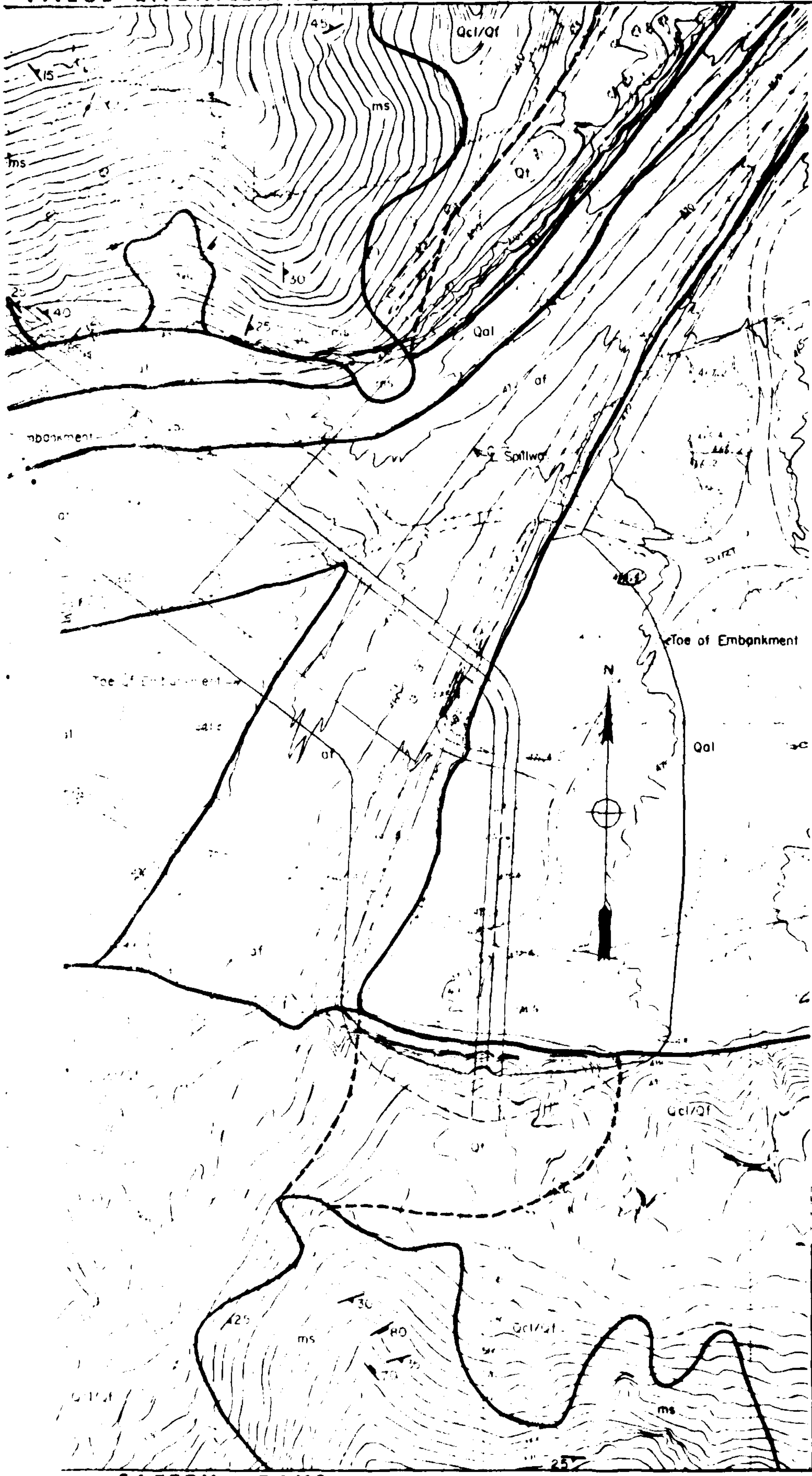
PLATE

II


ms
ms
ms
Qcl/Qf
Qcl
Qcl
Qal
af
℄ Embankment
Qal
DRY WASH
af
Toe of Embankment
Qal
af
SCATT BRUSH
Qal
af
Qcl/Qf
Qcl/Qf
Qcl/Qf
ms
ms
Qcl/Qf
ms

VALUE ENGINEERI.IG



LEGEND

af — Artificial fill, temporary sand levees

Qal — Active channel deposits consisting of well sorted, poorly graded coarse sand with cobbles, little to no fines. Inactive channel deposits are well graded, poorly sorted coarse sand and cobbles, boulders to 4' diameter, up to 8% fines

Qc — Slope wash and residual soil less than ... thick

Qcl/Qf — Undifferentiated slope wash and tributary fan deposits; stratified, well graded, poorly sorted with boulders ... diameter. Thickness varies from ... to ...

Qf — ...der fan deposits. Moderately stratified, angular to subangular metamorphic rock fragments ranging from gravel to 12" cobbles. Matrix consists of ... coarse sand to fine, moderately ... consolidated silty sand

ms — Undifferentiated metamorphics. Well layered metasediments including marbles, quartzites ... injected by metamorphic ... granites and vein quartz

Approximate contact between geologic units, dashed where inferred ... gradations

Strike and dip of joint

Strike and dip of foliation

Strike and dip of shear zone

NOTES

1. Description ... alluvium are based on field observations
2. See plate C ... plan of exploration

20 40 60

SYMBOL	DESCRIPTIONS	DATE	APP

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WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

SITE GEOLOGY MAP

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DATE APPROVED

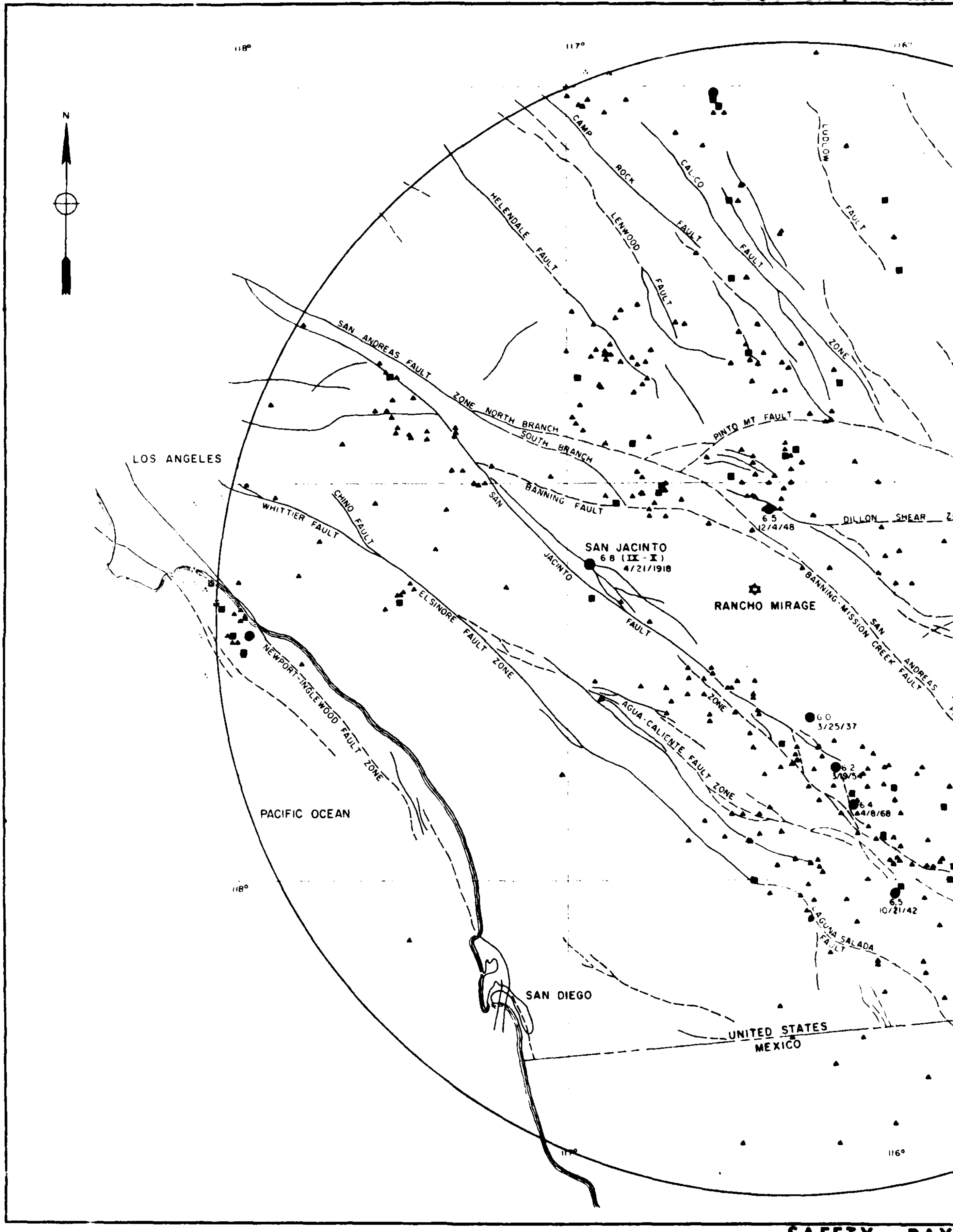
SPEC NO DACW09-____-B-____

DISTRICT FILE NO

SAFETY PAYS

PLATE C-

II


N
118°
117°
LOS ANGELES
PACIFIC OCEAN
SAN DIEGO
SAN JACINTO
6.8 (IX - X)
4/21/1918
RANCHO MIRAGE
CAMP ROCK FAULT
CALICO FAULT ZONE
LENWOOD FAULT
HELENDALE FAULT
LUDLOW FAULT
SAN ANDREAS FAULT ZONE NORTH BRANCH
SOUTH BRANCH
BANNING FAULT
PINTO MT FAULT
DILLON SHEAR
6.5
12/4/48
WHITTIER FAULT
CHINO FAULT
SAN JACINTO FAULT ZONE
BANNING-MISSION CREEK FAULT
SAN ANDREAS
ELSINORE FAULT ZONE
NEWPORT-INGLEWOOD FAULT ZONE
AGUA CALIENTE FAULT ZONE
6.0
3/25/37
6.4
4/8/68
6.5
10/21/42
LAGUNA SALADA FAULT
UNITED STATES
MEXICO
116°

I

LEGEND

- ▲ Earthquake with magnitude 4.0 through 4.9
- ■ Earthquake with magnitude 5.0 through 5.9
- ● Earthquake with magnitude greater than 6.0
- ✡ Location of project area
- — — Surface trace of fault

NOTES:

1. Location of fault traces are approximate a: are shown only to establish possible relations– to plotted epicenters.
2. All epicenters shown were instrumentally recorded between 1932 and 1979, except f the 1918 San Jacinto earthquake which is the closest large historic earthquake to th Rancho Mirage site.
3. The magnitude and date of occurrence of the 6+ magnitude events within 50 miles of the site are shown. An intensity valu for the San Jacinto event is also show

SYMBOL	DESCRIPTIONS	DATE
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CHANNEL AND DEBRIS BASIN

EPICENTER MAP
4+ MAGNITUDE WITHIN 50 MILES
1932 TO 1979

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DISTRICT FILE NO.

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II

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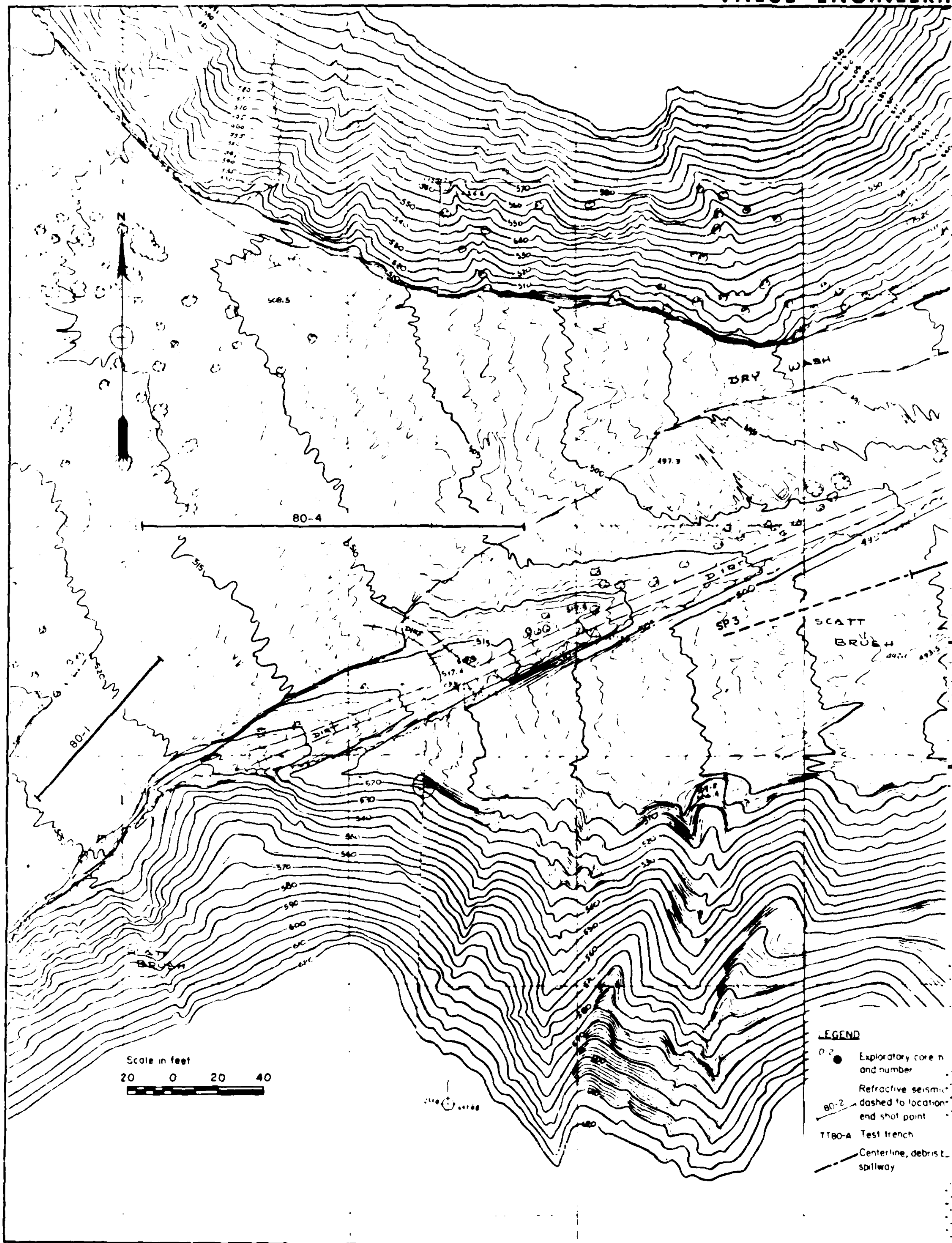
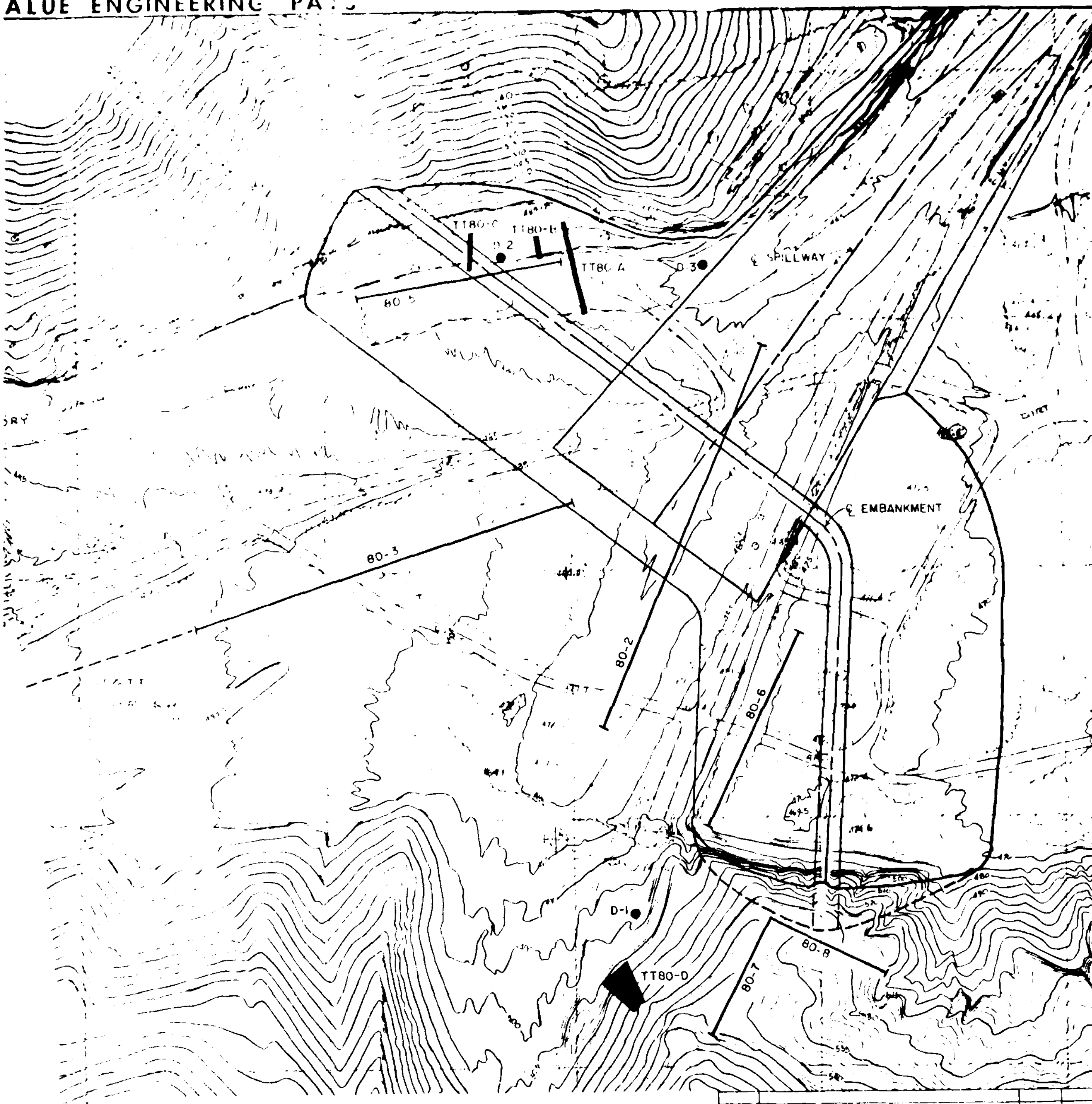


SAFETY PA

ALUE ENGINEERING PA



NOTES

1. See plate C-1 for regional geologic map
2. See plate C-2 for geologic map of the greater Rancho Mirage area
3. See plates C-6 and C-7 for Time-distance curves and profiles for refractive seismic survey lines
4. See plate C-8 for logs of drill holes
5. See plate C-9 for logs of test trenches and geologic profile along embankment centerline

SYMBOL	DESCRIPTIONS	DATE	APPROV.

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WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

SITE EXPLORATION MAP

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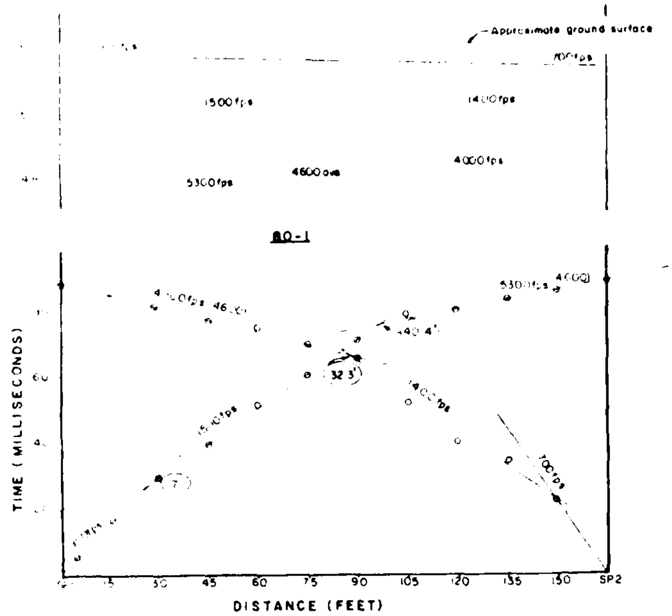
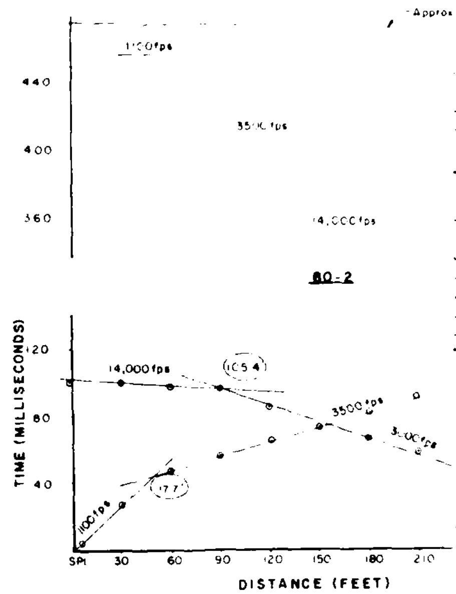
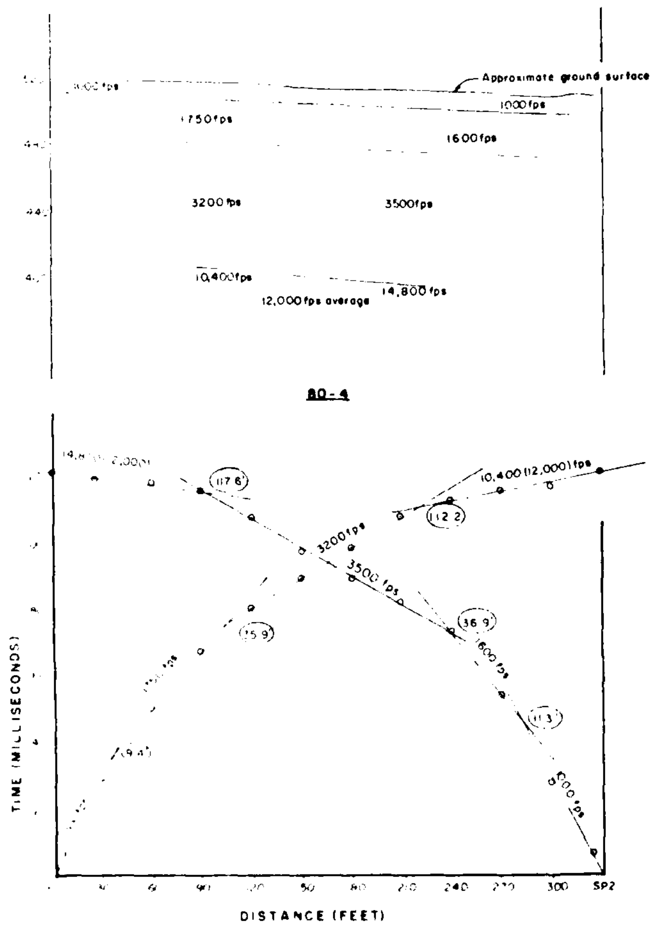


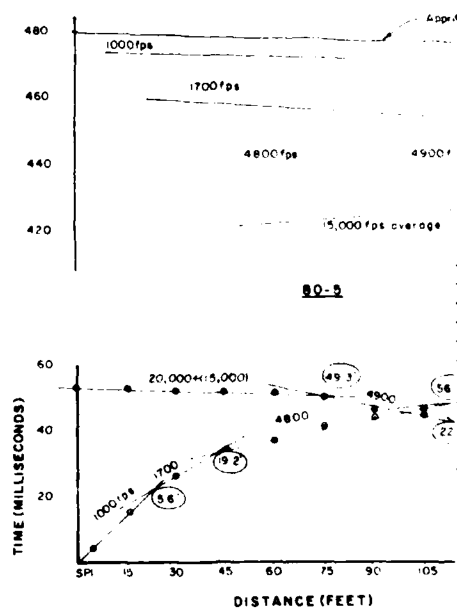
Approximate ground surface
1500 fps
1400 fps
5300 fps
4600 ave
4000 fps
BQ-1
5300 fps
1400 fps
(32.3')
TIME (MILLISECONDS)
DISTANCE (FEET)
15
30
45
60
75
90
105
120
135
150
SP2



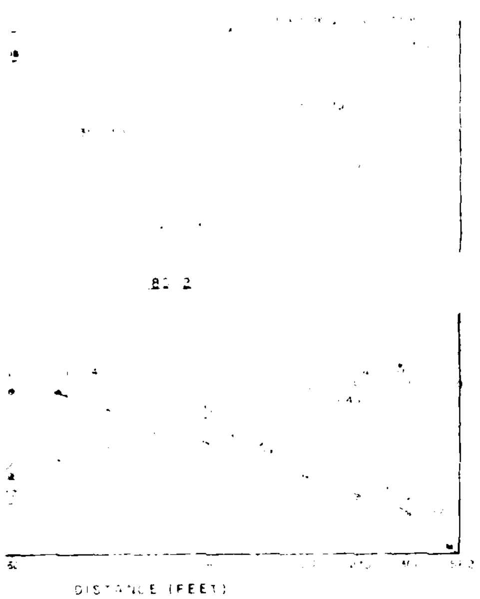
440
400
360
14,000 fps
BQ-2
120
80
40
14,000 fps
(105.4)
3500 fps
1100 fps
(17.7)
TIME (MILLISECONDS)
SP1
30
60
90
120
150
180
210
DISTANCE (FEET)

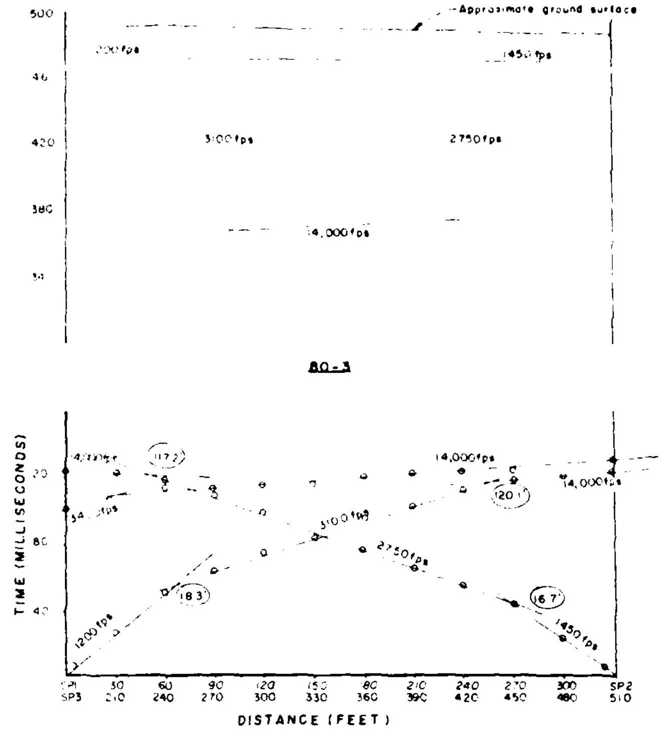


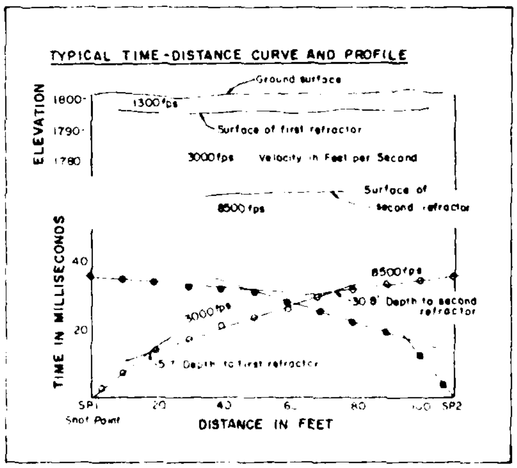
Approximate ground surface
1750 fps
1000 fps
1600 fps
3200 fps
3500 fps
10,400 fps
12,000 fps average
14,800 fps
BQ-4
(117.6)
10,400 (12,000) fps
(112.2)
3200 fps
3500 fps
(36.9)
1600 fps
(15.9)
(9.4)
(11.3)
TIME (MILLISECONDS)
240
300
SP2
DISTANCE (FEET)

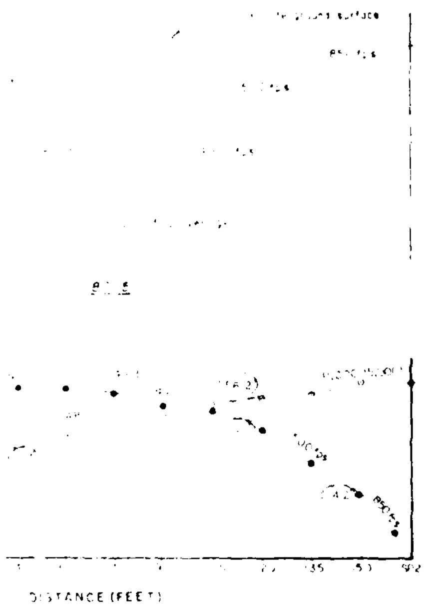


480
460
440
420
1000 fps
1700 fps
4800 fps
15,000 fps average
BQ-5
60
40
20
20,000+(15,000)
(49.3)
4900
4800
1700
1000 fps
(19.2)
(5.6)
TIME (MILLISECONDS)
SP1
15
30
45
60
75
90
105
DISTANCE (FEET)









NOTES

1. See plate C-5 for location of all refractive seismic lines.
2. See plate C-7 for time-distance curves and profiles for lines 80-6 thru 80-8.
3. Horizontal and vertical scales vary.

SYMBOL	DESCRIPTIONS	DATE	APPROV.

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WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN
REFRACTIVE SEISMIC SURVEY LINES
TIME-DISTANCE CURVES AND PROFILES
LINE 80-1 THRU 80-5

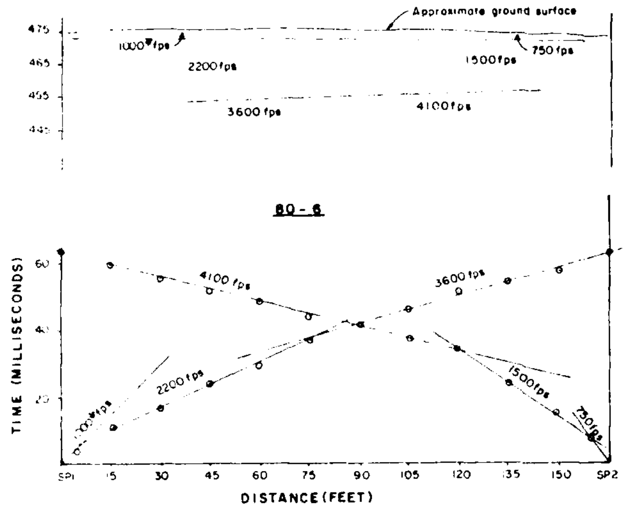
SUBMITTED BY: DATE APPROVED: SPEC NO. DACW09-____-B-____ SHEET

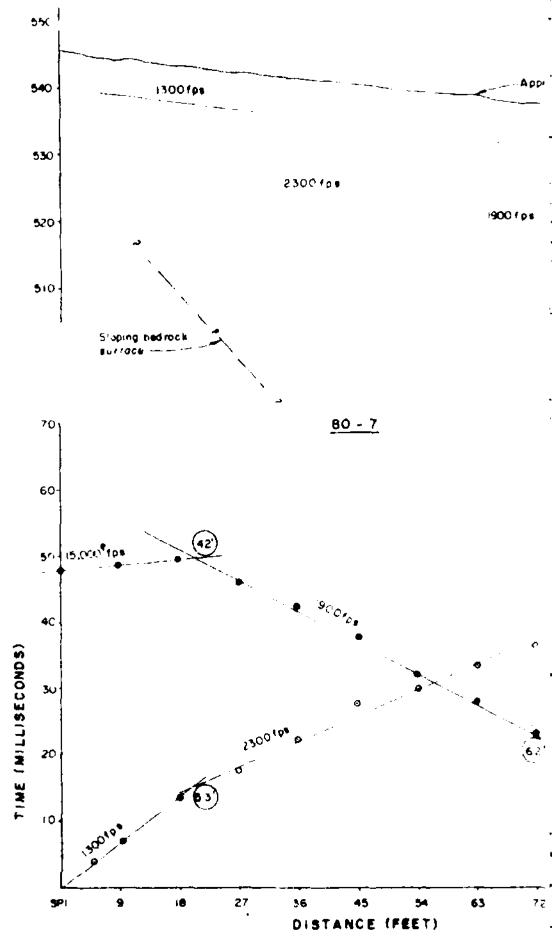
DISTRICT FILE NO.

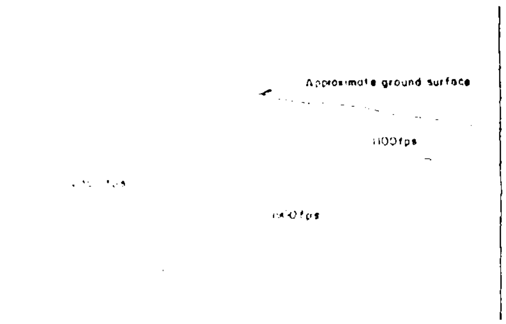
SAFETY PAYS

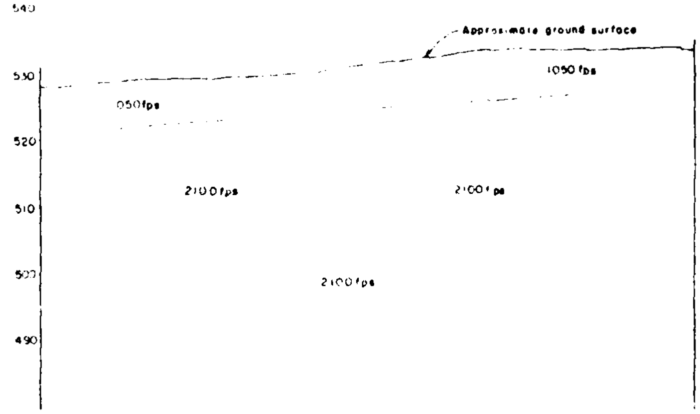
PLATE C-

II


Approximate ground surface
475
465
455
445
1000 fps
750 fps
2200 fps
1500 fps
3600 fps
4100 fps
80 - 6
TIME (MILLISECONDS)
60
40
20
4100 fps
3600 fps
2200 fps
1500 fps
750 fps
SP1
15
30
45
60
75
90
105
120
135
150
SP2
DISTANCE (FEET)

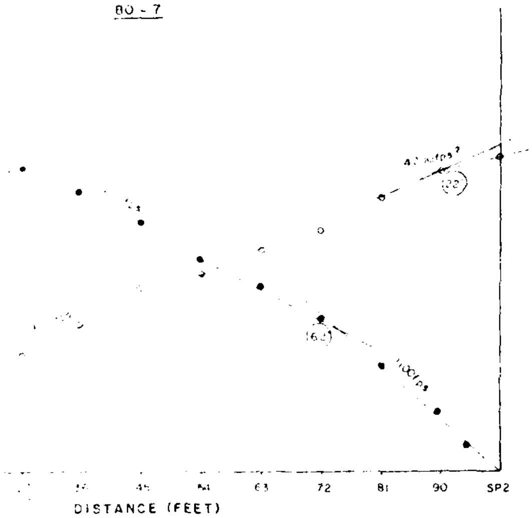

550
540
530
520
510
1300 fps
2300 fps
1900 fps
Sloping bedrock surface
80 - 7
70
60
50
40
30
20
10
TIME (MILLISECONDS)
15,000 fps
42'
900 fps
2300 fps
1300 fps
SP1
9
18
27
36
45
54
63
72
DISTANCE (FEET)

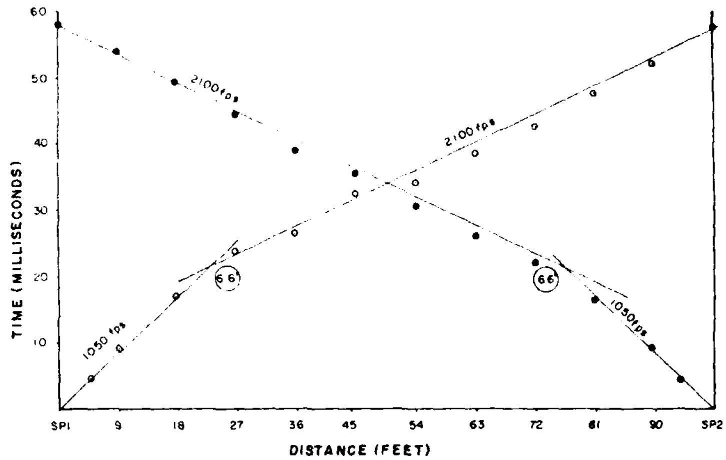




80 - 7

80 - 8





NOTES:

1 See plate C-6 for typical time-distance curve and profile and notes

2 Horizontal and vertical scales vary

SYMBOL	DESCRIPTIONS	DATE

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY:

CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN
REFRACTIVE SEISMIC SURVEY LINES
TIME-DISTANCE CURVES AND PROFILE
LINE 80-6 THRU 80-8

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW 09- ___ B- ___

DISTRICT FILE NO.

D-1

Description

... 22.3 ALLUVIUM: Terrace deposits; poorly sorted gravelly sand to sandy gravel with cobbles. Rotary drilled with tri-cone rock bit vertically through alluvium to bedrock refusal.

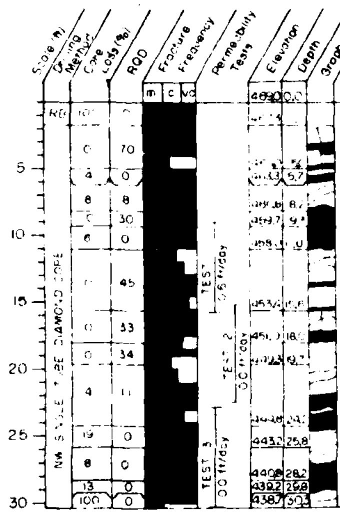
22.3 - 37.8 UNDIFFERENTIATED METAMORPHICS: Interlayered diorite, pegmatitic granite, quartz schist and gneiss. Weathered, moderately hard and highly fractured except for the garnet-bearing pegmatitic granite which is moderately weathered, moderately fractured and hard. Diorite and quartz schist break readily into rubble zones. Joints/fractures dip 30-60° and approximately ... 3 contain minor $CaCO_3$ and Fe_2O_3 coatings. Foliations parallel dip.

Drilled 16 - 18 December 1980.

PRESSURE TEST DATA

Test Number	1						
Depth Interval	24.0 TO 37.8						
Pressure PSI	5	10	15	20	15	10	5
Q GPM	05	07	11	15	10	07	06
K ft/day	01	01	02	02	02	01	01

D-3



... UNDIFFE...

Interlayered diori... granite. Marble, ... weathered and ha... ranging from clos... gneiss are weath... tures closely to ... into rubble zones ...

Fractures/joints d... mately 3-4 conta... vertical fractures ... 1.7-5.0'. Thickne... 0.3 to 1.2'.

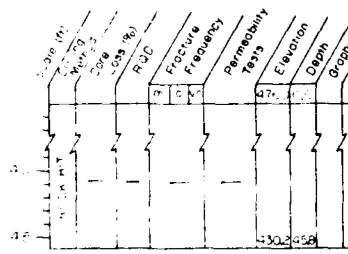
Drilled 19 - ... Dec...

PRESSURE TEST DATA

Test Number	1				
Depth Interval	8.1 TO 15.6				
Pressure PSI	5	10	5	—	—
Q GPM	1.0	1.4	1.0	—	—
k ft/day	0.6	0.6	0.6	—	—

Test Number	2				
Depth Interval	15.3 TO 22.4				
Pressure PSI	10	5	20	15	10
Q GPM	0.0	0.0	0.0	0.0	0.0
k ft/day	0.0	0.0	0.0	0.0	0.0

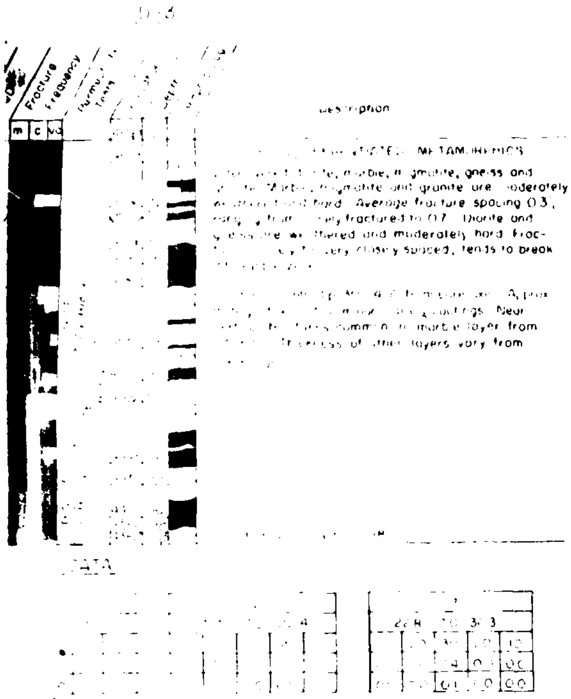
D-2



Description

0.0 - 45.8 ALLUVIUM: Channel deposits; gravelly sand and clay. A few cobbles less than 6". Rotary drilled with tri-cone rock bit vertically through alluvium to bedrock refusal.

Drilled 18 December 1980.



NOTES

1 See plate C-5 for approximate locations of drill holes
2 Rock bit is abbreviated RB on D-3 log
3 RQD (rock quality designation) is defined as the total length of individual core sticks per drill run greater than [illegible] mm in length divided by the length of the drill run and expressed as a percent
4 Fracture frequency is the number of fractures per foot and is designated as vc (very closely fractured) – greater than 6 fractures per foot; c (closely fractured) – 4 to 6 fractures per foot and m (moderately fractured) – less than 4 fractures per foot
5 Permeabilities are given as k values in ft/day as listed in pressure test data tables beneath the core logs
6 Ground surface elevations are not surveyed
7 The graphic log represents a generalized fracture pattern. Solid black areas indicate rubble zones

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
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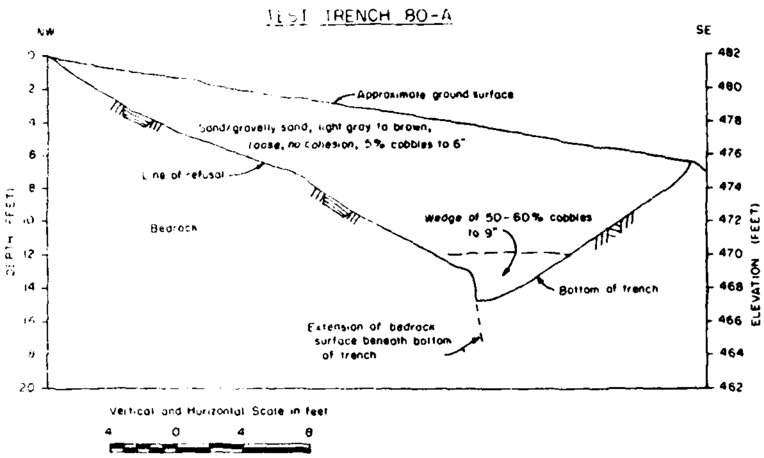
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

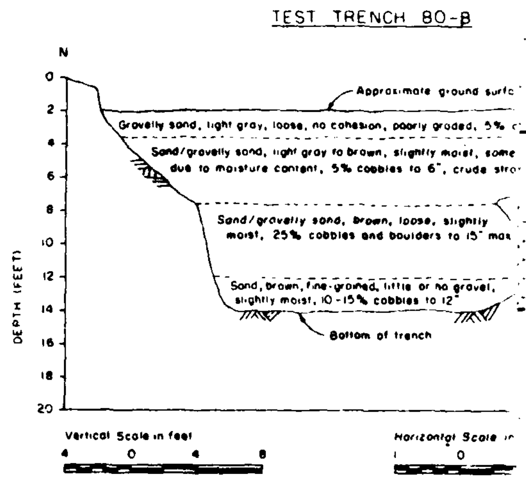
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

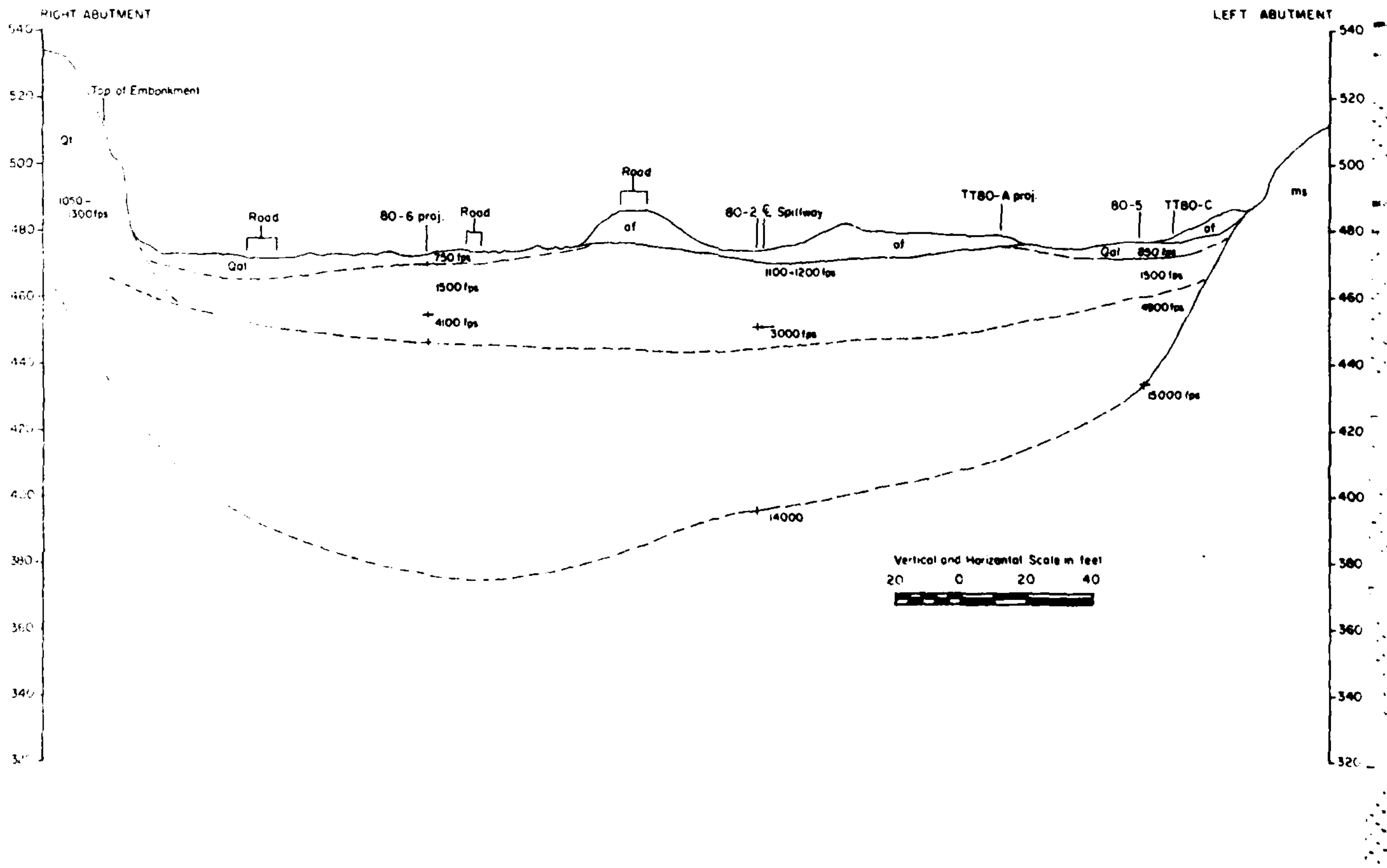
LOGS OF DRILL HOLES AND
PRESSURE TEST RESULTS

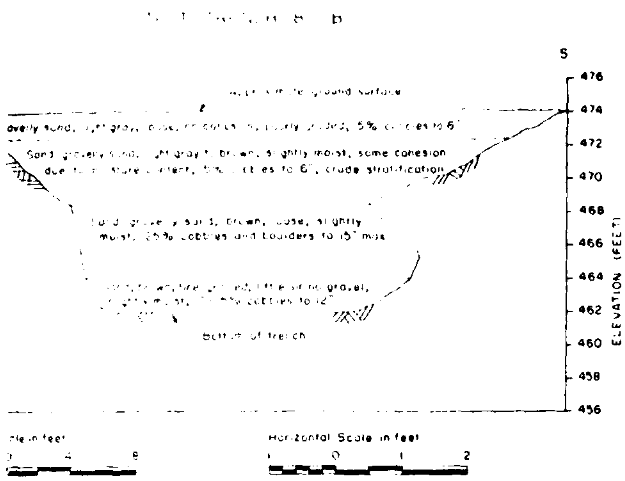
SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-____-B-____ SHEET
DISTRICT FILE NO.

II

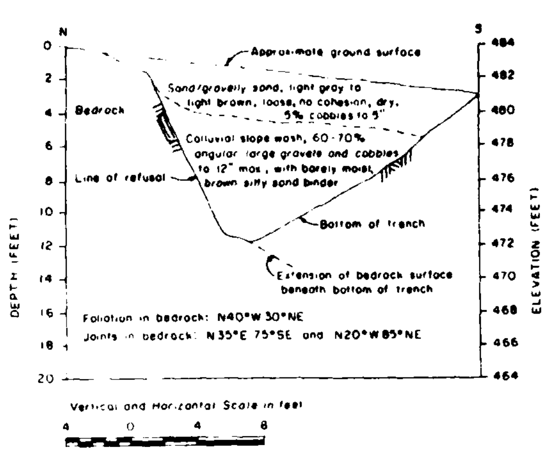

TEST TRENCH 80-A
NW
SE
Approximate ground surface
Sand/gravelly sand, light gray to brown, loose, no cohesion, 5% cobbles to 6"
Line of refusal
Bedrock
Wedge of 50-60% cobbles to 9"
Bottom of trench
Extension of bedrock surface beneath bottom of trench
DEPTH (FEET)
ELEVATION (FEET)
Vertical and Horizontal Scale in feet
4 0 4 8


TEST TRENCH 80-B
N
Approximate ground surfc
Gravelly sand, light gray, loose, no cohesion, poorly graded, 5% c
Sand/gravelly sand, light gray to brown, slightly moist, some
due to moisture content, 5% cobbles to 6", crude stro
Sand/gravelly sand, brown, loose, slightly moist, 25% cobbles and boulders to 15" max
Sand, brown, fine-grained, little or no gravel, slightly moist, 10-15% cobbles to 12"
Bottom of trench
DEPTH (FEET)
Vertical Scale in feet
4 0 4 8
Horizontal Scale in

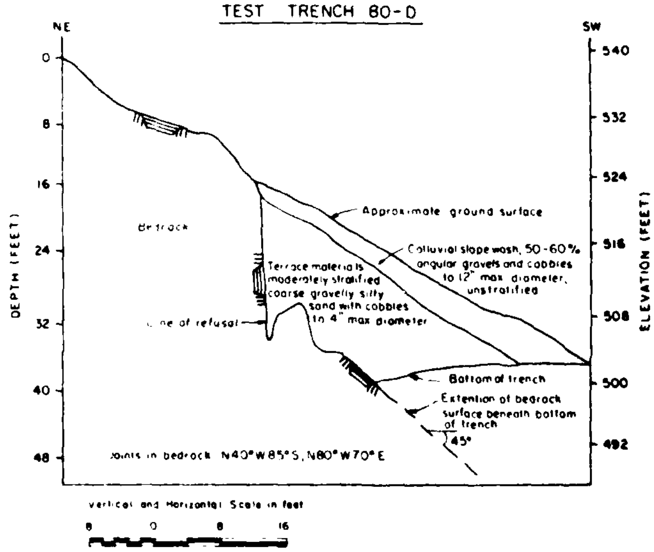

RIGHT ABUTMENT
LEFT ABUTMENT
Top of Embankment
Qt
1050-1300 fps
Road
80-6 proj.
Road
Road
af
80-2 ℄ Spillway
TT80-A proj.
80-5
TT80-C
ms
Qal
750 fps
1500 fps
4100 fps
1100-1200 fps
3000 fps
850 fps
1500 fps
4800 fps
15000 fps
14000
Vertical and Horizontal Scale in feet
20 0 20 40

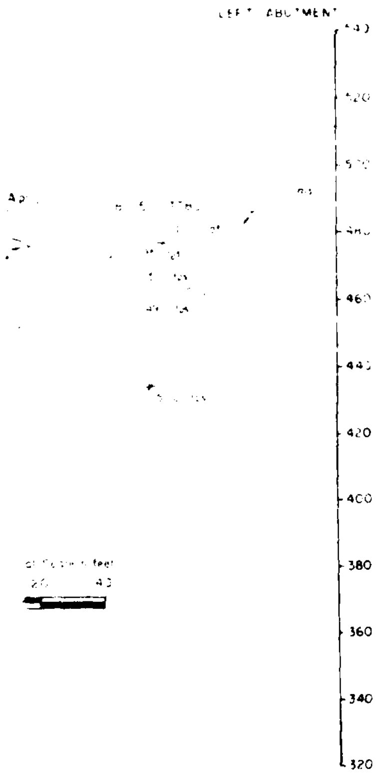


TEST TRENCH 80-C



TEST TRENCH 80-D





NOTES

1. See plate C-3 for legend
2. See plate C-5 for locations of geologic profile beneath embankment centerline, refractive seismic survey lines and legend
3. See plates C-6 and C-7 for Time-Distance Curves and Profiles for refractive seismic surveys
4. See plate C-8 for logs of drill holes
5. Numbers (3100) associated with seismic survey lines are velocitys in feet per second
6. Soil descriptions on logs are from visual examination in the field

SYMBOL	DESCRIPTIONS	DATE	APP.

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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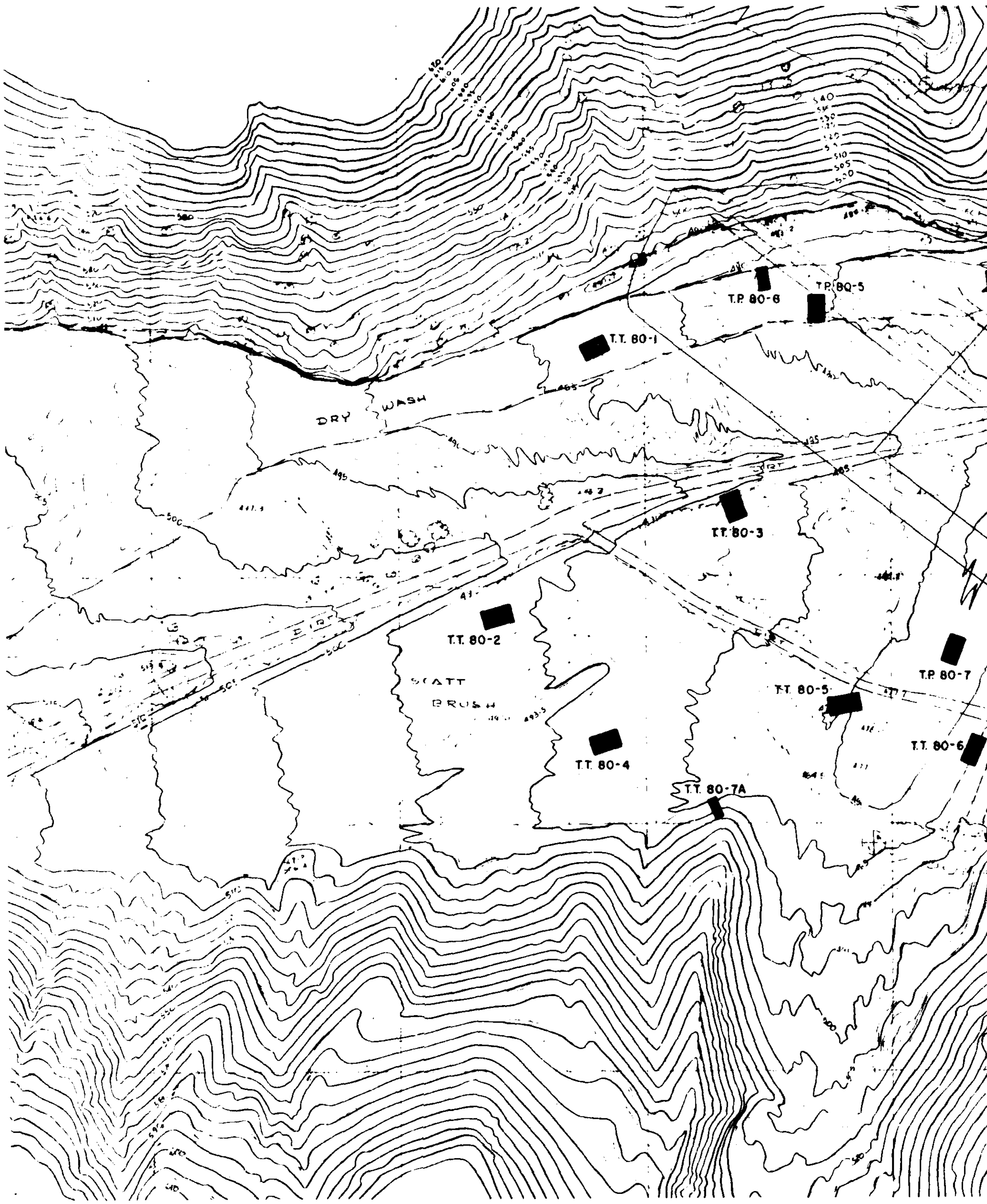
DRAWN BY:
CHECKED BY:

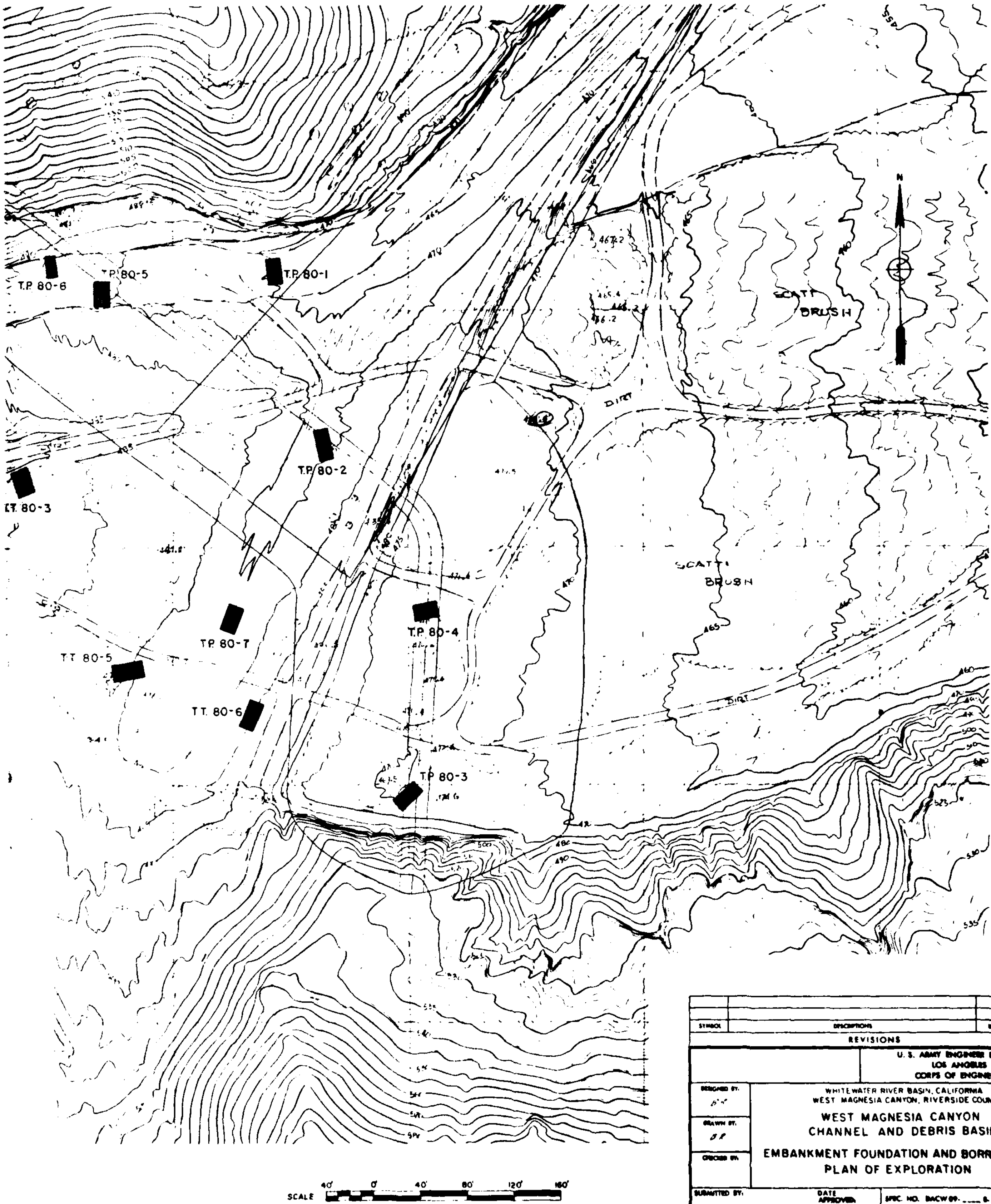
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

TEST TRENCH PROFILES
AND
GEOLOGIC CROSS SECTION

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-
DISTRICT FILE NO.


T.P. 80-6
T.P. 80-5
T.T. 80-1
DRY WASH
T.T. 80-3
T.T. 80-2
SCATT
BRUSH
T.T. 80-4
T.T. 80-7A
T.T. 80-5
T.P. 80-7
T.T. 80-6



SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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LOS ANGELES
CORPS OF ENGINEERS

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CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

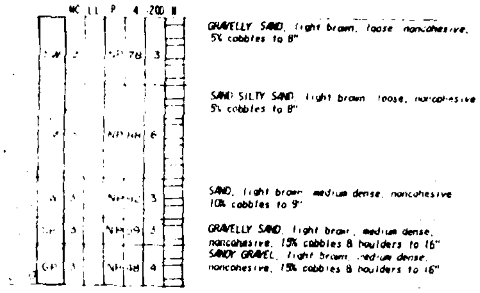
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

EMBANKMENT FOUNDATION AND BORROW AREAS
PLAN OF EXPLORATION

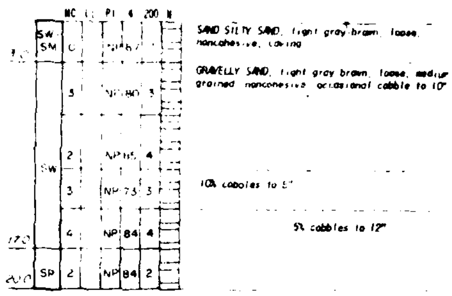
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-B-____
SHEET
DISTRICT FILE NO.

II

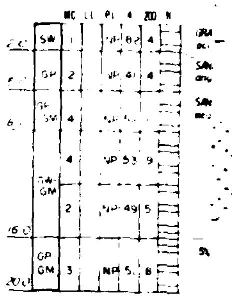
TP 80-1



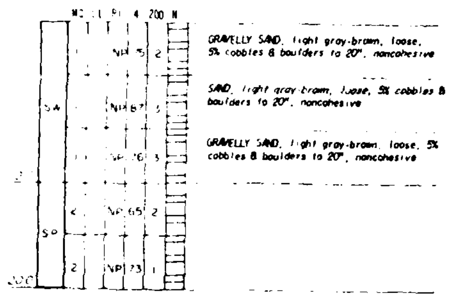
TP 80-2



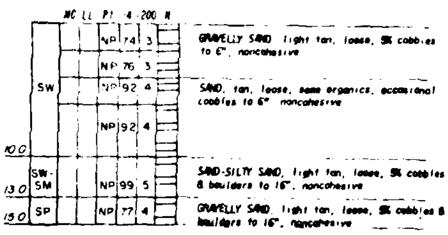
TP 80-3



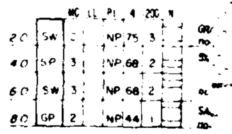
TP 80-4



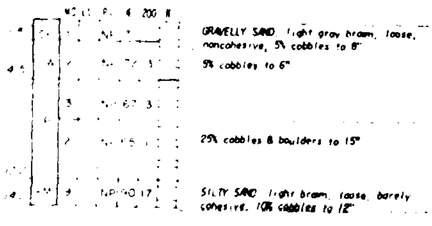
TP 80-4A



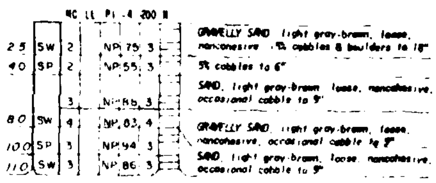
TP 80-5



TP 80-6



TP 80-7



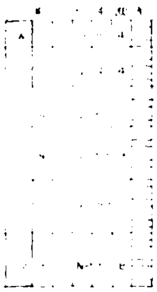
UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than no. 200 sieve size.	GRAVELS More than half of coarse fraction is larger than no. 4 sieve size.	Clean gravels	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS More than half of coarse fraction is smaller than no. 4 sieve size.	Clean sands	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands, gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS More than half of material is smaller than no. 200 sieve size.	SILTS AND CLAYS	Low liquid limit	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
		High liquid limit	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
Highly organic soils			Pt	Peat and other highly organic soils.

NOTES:

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U. S. Standard.
3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart (Table VI, Military Standard 619A), and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.
4. For a complete description of the Unified Soil Classification System, see "Military Standard 619A" dated 20 March 1962.

TP 80-3



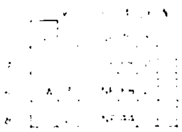
GRAVELLY SAND: light gray brown, loose, occasional cobble to 6"

SANDY GRAVEL: light brown, medium dense [illegible] gravel, 5% cobbles to 10"

SANDY GRAVEL SILTY SANDY GRAVEL, light brown, medium dense, 25% cobbles & boulders to 18"

5% cobbles to 6"

TT 80-5



GRAVELLY SAND: light gray-brown, loose, noncohesive, 5% cobbles & boulders to 15"

5% cobbles to 8"

occasional cobble to 6"

SANDY GRAVEL: light gray brown, loose, noncohesive, occasional cobble to 6"

LEGEND

MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
LL	LIQUID LIMIT.
PI	PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT).
NP	NONPLASTIC
-4	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 4 SIEVE.
-200	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 200 SIEVE.
N	NUMBER OF BLOWS OF A 140-POUND DROPHAMMER FALLING 30 INCHES REQUIRED TO DRIVE A SAMPLING SPOON ONE FOOT. OUTSIDE DIAMETER OF SPOON IS 2 INCHES; INSIDE DIAMETER IS 1 3/8 INCHES. PROCEDURE IS CALLED STANDARD PENETRATION TEST.
W	DEPTH TO WATER
TP 80-2	LOCATION AND NUMBER OF TEST PIT
TT 80-5	LOCATION AND NUMBER OF TEST TRENCH

NOTES

1. ALL TEST PITS AND TEST TRENCHES WERE EXCAVATED WITH A GRADALL OR BACKHOE IN SEPTEMBER THROUGH NOVEMBER, 1980
2. NO GROUND WATER WAS ENCOUNTERED
3. SEE PLATE C-10 FOR LOCATION OF TEST PITS.

SYMBOL	DESCRIPTIONS	DATE

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

EMBANKMENT FOUNDATION SOIL LOG

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO.

SAFETY PAYS

PLAT[illegible]

II

TT 80-1

Depth		MC	LL	PI	-4	-200	N	Description
	SP	2		NP	89	3		SAND Light gray-brown, loose, occasional cobbles & boulders to 14", noncohesive, caving
		3		NP	67	3		GRAVELLY SAND Light gray-brown, loose, occasional cobbles & boulders to 14", noncohesive, caving
12.0		3		NP	97	3		SAND Light gray-brown, loose, noncohesive, gravel to 1"
16.0	SW	3		NP	86	3		

TT 80-2

Depth		MC	LL	PI	-4	-200	N	Description
3.0'	SM			NP	95	15		SILTY SAND Light gray-brown, dry, loose, noncohesive, caving
7.0'	SW-SM			NP	81	7		GRAVELLY SAND-SILTY GRAVELLY SAND Light, gray brown, dry, loose, noncohesive, gravel to 3"
9.0'	SP-SM	3		NP	92	6		SAND-SILTY SAND Light gray-brown, loose, noncohesive
12.0'	SW	2		NP	89	3		SAND Light gray-brown, loose, noncohesive

Depth	
7.0'	SW
15.0'	SP

TT 80-5

Depth		MC	LL	PI	-4	400	N	Description
3.0'	SW	0		NP	80	4		GRAVELLY SAND Light gray-brown, loose, noncohesive, 5% cobbles to 4"
7.0'	SP	1		NP	90	3		SAND Light gray-brown, loose, noncohesive, 5% cobbles to 6"
12.0'	SW	3		NP	72	3		GRAVELLY SAND Light gray-brown, loose, noncohesive, 5% cobbles to 6"
16.0'	SP-SM	6		NP	61	11		GRAVELLY SAND-SILTY GRAVELLY SAND Light gray-brown, loose, noncohesive, 10% cobbles & boulders to 15"

TT 80-6

Depth		MC	LL	PI	-4	-200	N	Description
4.0'	SP	1		NP	63	2		GRAVELLY SAND Light gray-brown, loose, noncohesive, 5% cobbles to 4"
9.0'	GW	2		NP	42	2		SANDY GRAVEL Light gray-brown, loose, noncohesive, 3% cobbles to 4"
12.0'	SW	3		NP	91	3		SAND Light gray-brown, loose, noncohesive, 10% cobbles to 9"
15.0'	SP	2		NP	93	3		5% cobbles to 6"

TT80-3

Depth	Class	MC	LL	PI	-4	200	N	Description
	SW	1		NP	85	4		GRAVELLY SAND Light gray-brown, loose, caving, occasional cobble to 6"
		1		NP	79	3		
[illegible]	SP	1		NP	56	1		10% cobbles to 10"
15.0'		2		NP	52	1		5% cobbles to 8"

TT80-4

Depth	Class	MC	LL	PI	4	-400	N	Description
3.0'	SW	1		NP	87	4		SAND Light gray-brown, loose, caving noncohesive, 5% cobbles to 8"
7.0'	SP-SM	1		NP	71	5		GRAVELLY SAND SILTY GRAVELLY SAND Light gray-brown, loose, noncohesive 5% cobbles to 10"
10.0'	GP	2		NP	42	4		SANDY GRAVEL Light gray brown, loose noncohesive, 10% cobbles to 7"
14.0'	SW	2		NP	75	4		GRAVELLY SAND Light gray brown, loose 5% cobbles to 6"

TT80-7A

Depth	Class	MC	LL	PI	4	200	N	Description
	SW-SM			NP	59	9		GRAVELLY SAND SILTY GRAVELLY SAND Light brown, dry, medium dense, angular gravel, 25% cobbles & boulders to 20"

NOTES

1. SEE PLATE C-10 FOR LOCATION OF TEST TRENCHES
2. ALL TEST TRENCHES WERE EXCAVATED WITH A GRADALL OR BACKHOE IN SEPTEMBER THROUGH NOVEMBER, 1980
3. SEE PLATE C-11 FOR LEGEND

SCALE 1 IN = 5 FT

SYMBOL	DESCRIPTIONS	DATE	APPROV

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY

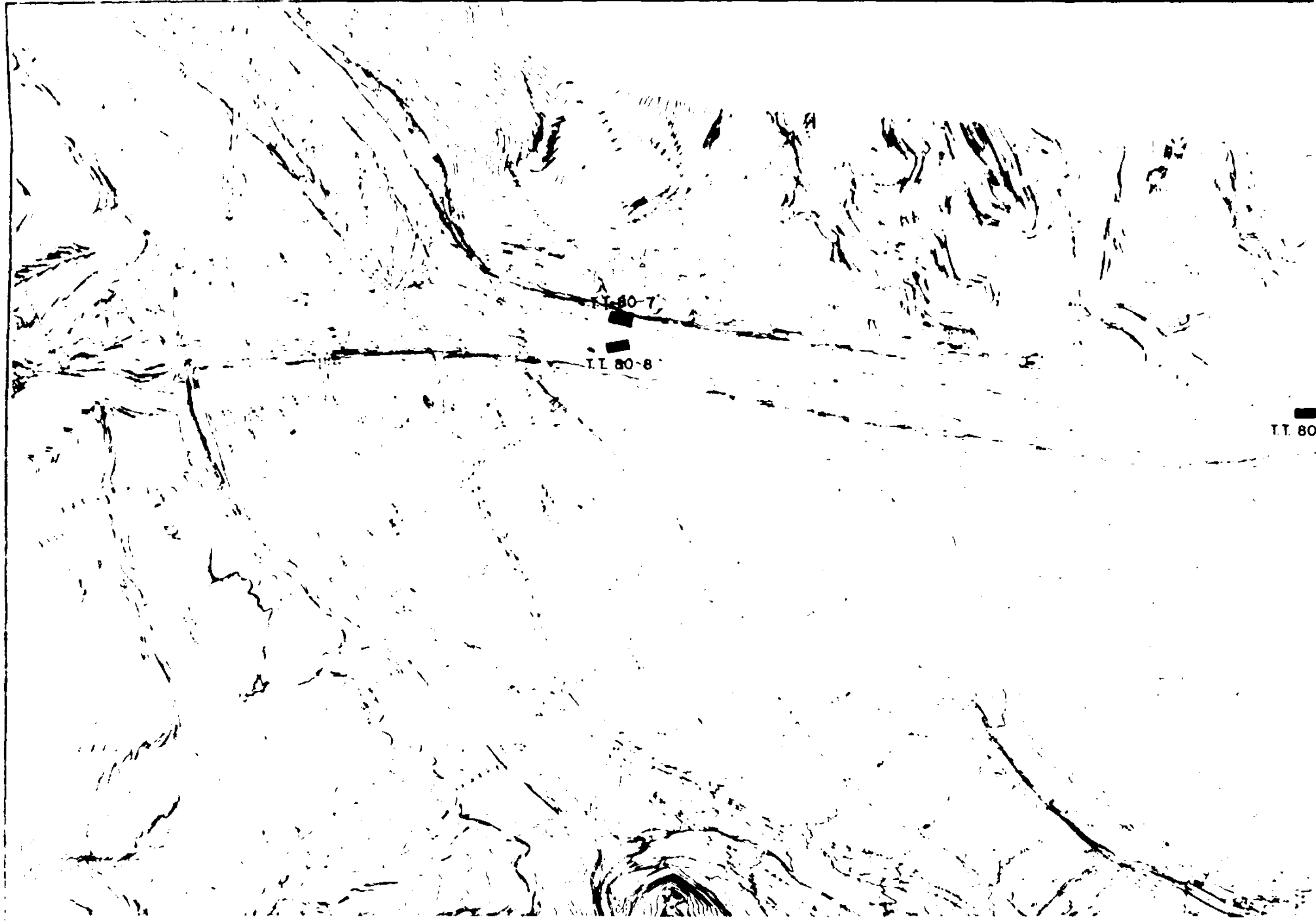
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

BORROW AREA SOIL LOGS

SUBMITTED BY:
DATE APPROVED:
SPEC NO. DACW09-
DISTRICT FILE NO

PLATE C-



T T 80-7

Depth	Class	MC	LL	PI	-4	-200	N	Description
0–3.0	SP			NP	60	1		GRAVELLY SAND Light gray-brown, loose, noncohesive, 10 % cobbles to 4"
		5		NP	72	5		GRAVELLY SAND-SILTY GRAVELLY SAND Light gray-brown, loose, noncohesive, gravel to 3".
–15.0	SW-SM	3		NP	91	5		SAND-SILTY SAND Light gray-brown, loose, noncohesive, gravel to 1".

T.T. 80-8

Depth	Class	MC	LL	PI	-4	-200	N	Description
	GP	1		NP	50	3		SANDY GRAVEL Light grey-brown, loose, noncohesive.
5.0'	GP	2		NP	45	2		15 % cobbles to 6"
7.0'	SW-SM	3		NP	97	8		SAND-SILTY SAND Light grey-brown, loose, noncohesive.
15.0'	SW	2		NP	71	4		GRAVELLY SAND Light grey-brown, loose, noncohesive, 5 % cobbles to 4"

Depth	Class
3.0'	SP
4.0'	SW
8.0	SM
10.0	GP-GM

T T 80-13

Depth	Class	MC	LL	PI	-4	-200	N	Description
4	GP			NP	37	1		SANDY GRAVEL Light gray-brown, barely moist, loose, noncohesive, 15 % cobbles to 12".
	SP			NP	91	4		SAND Light brown, moist, medium dense, noncohesive, occasional cobble to 5"

T T 80-16

Depth	Class	MC	LL	PI	-4	-200	N	Description
3.0	SW			NP	78	2		GRAVELLY SAND Light gray-brown, dry, loose, noncohesive, 10 % cobbles to 6"
6.0'	SP-SM			NP	66	11		GRAVELLY SAND-SILTY GRAVELLY SAND Light brown, moist, medium dense, 5 % cobbles to 8"
9.0	SM			NP	81	15		SILTY GRAVELLY SAND Light brown, moist, medium dense, 5 % cobbles to 8".
10.0	SW-SM			NP	90	5		SAND-SILTY SAND Light brown, moist, loose, noncohesive, gravel to 1"

TT 80-9

Depth	Symbol	MC	LL	PI	-4	-200	N	Description
	SP	[illegible]		NP	69	3		GRAVELLY SAND Light gray-brown, loose, noncohesive, 10 % cobbles to 6"
3.0	SW	[illegible]		NP	92	4		SAND Light brown, medium dense
4.[illegible]	SM	[illegible]		NP	83	[illegible]5		GRAVELLY SILTY-SAND light brown, medium dense, noncohesive, occasional cobble to 8"
	[illegible]-SM	3		NP	53	7		SANDY GRAVEL-SILTY SANDY GRAVEL Light brown, medium dense

TT 80-10

Depth	Symbol	MC	LL	PI	-4	-200	N	Description
1.5	SW-SM	3		NP	60	8		GRAVELLY SAND-SILTY GRAVELLY SAND L[illegible] brown medium dense,
3.5	SP-SM	4		NP	57	10		occasional cobble to 4"
5.0	GW-GM	3		NP	46	7		SANDY GRAVEL SILTY SANDY GRAVEL Ligh[illegible] brown, medium dense, 5 % cobbles to 6[illegible]
6.5	GP-GM	4		NP	47	6		
8.0'	SM	4		NP	7[illegible]	3		SILTY GRAVELLY SAND Light brown, med[illegible] dense, noncohesive, occasional co[illegible] to 5"

NOTES

1. THIS DRAWING TO BE USED FOR LOCATION OF TEST TRENCHES ONLY.
2. SEE PLATE C-11 FOR LEGEND.
3. ALL TEST TRENCHES WERE EXCAVATED WITH A GRADALL IN SEPTEMBER, 1980

SCALE 100 0 100' 200' 300' 400'

SYMBOL	DESCRIPTIONS
REVISIONS	

U. S. ARMY ENGINEER[illegible]
LOS ANGELES
CORPS OF ENGIN[illegible]

DESIGNED BY: B R
DRAWN BY: B K
CHECKED BY:

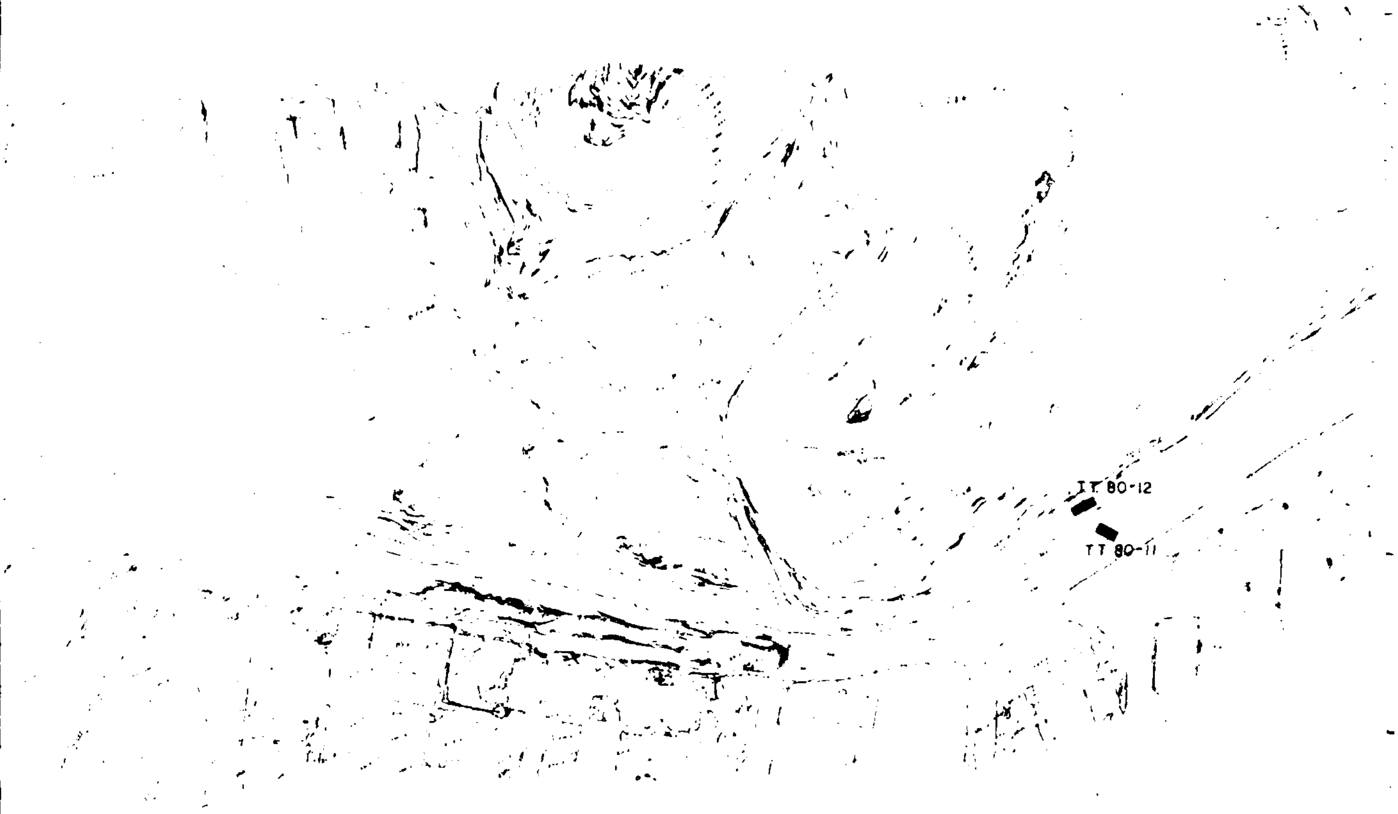
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COU[illegible]

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BAS[illegible]

UPPER CHANNEL REACH
PLAN OF EXPLORATION AND SOIL[illegible]

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO

II

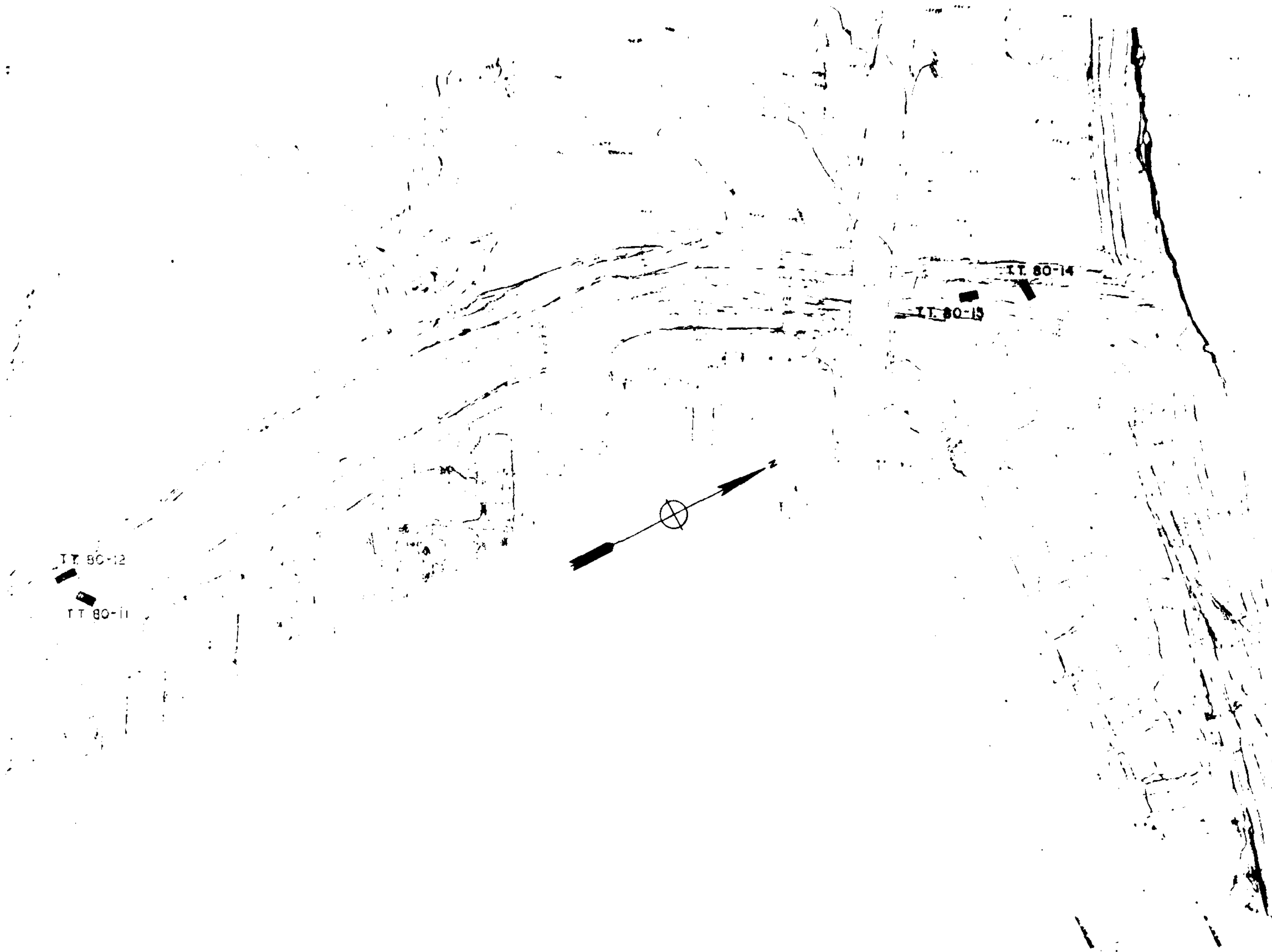


T T 80-11

MC	LL	PI	4	200	N	Class	Description
		NP	[illegible]9	6		SW-SM	GRAVELLY SAND-SILTY GRAVELLY SAND Light gray-brown, dry, medium dense, noncohesive, 10 % cobbles & boulders to 24".
		NP	70	5			
		NP	77	3		SW	GRAVELLY SAND Light gray-brown, dry, medium dense, noncohesive, 10 % cobbles & boulders to 24".
		NP	74	3			

T T 80-12

Depth	Class	MC	LL	PI	-4	-200	N	Description
2.0'	SW-SM			NP	76	5		GRAVELLY SAND-SILTY GRAVELLY SAND Light gray-brown, dry, medium dense, noncohesive, 25 % cobbles & boulders to 24".
4.0'	SP			NP	69	2		
6.0'	SW			NP	73	2		GRAVELLY SAND Light gray-brown, dry, medium dense, noncohesive, 25 % cobbles & boulders to 24".



T T 80-14

	MC	LL	PI	-4	-200	N	
SP			NP	59	1		GRAVELLY SAND Light gray-brown, dry, medium dense, 5 % cobbles to 12", noncohesive
			NP	58			moist
			NP	99	4		SAND Light gray-brown, moist, medium dense,
ML				100	69		SANDY SILT Light brown, moist, dense, moderately cohesive

T T. 80-15

Depth		MC	LL	PI	-4	-200	N	
	SM			NP	93	16		SILTY SAND Light gray-brown, dry, loose, noncohesive, 5 % cobbles to 4"
3.0'	CP			NP	43	1		SANDY GRAVEL Light gray-brown, dry, loose, noncohesive, 10 % cobbles to 4"
				NP	46	1		barely moist, 15 % cobbles to 6"
10.0'–11.0'	SM				87	24		SILTY SAND Brown, moist, fine grained, moderately cohesive.

NOTES

1. THIS DRAWING TO BE USED FOR LOCATION OF TEST TRENCHES ONLY.
2. SEE PLATE C-11 FOR LEGEND
3. ALL TEST TRENCHES WERE EXCAVATED WITH A GRADALL IN SEPTEMBER, 1980

SCALE 100' 0 100' 200' 300' 400'

SYMBOL	DESCRIPTIONS	DATE
REVISIONS		

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

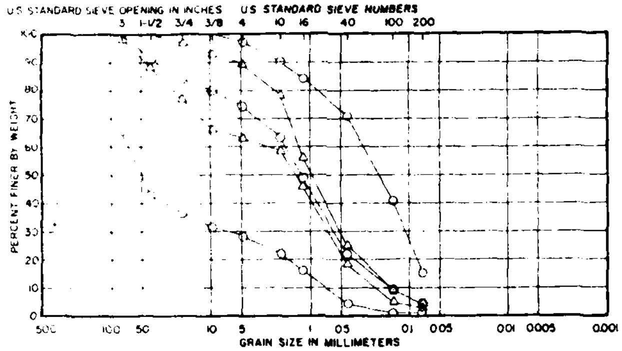
DESIGNED BY: B.R.
DRAWN BY: B.R.
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

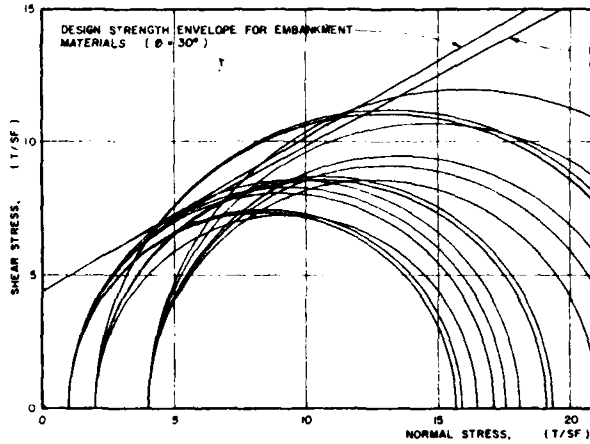
LOWER CHANNEL REACH
PLAN OF EXPLORATION & SOIL LOGS

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-__-__
DISTRICT FILE NO.

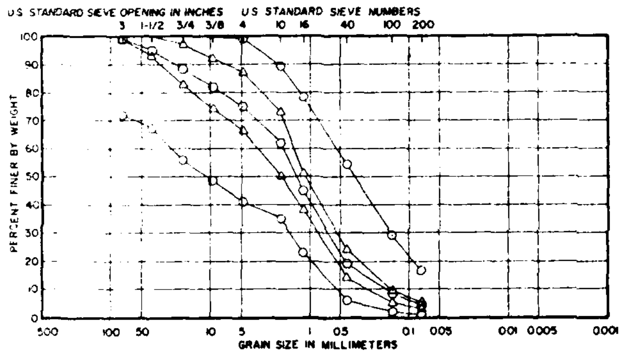


EMBANKMENT FILL MATERIALS

○ — UPPER AND LOWER GRADATION LIMITS
△ — UPPER AND LOWER QUARTILES
○ — MEAN GRADATION



CONSOLIDATED-UNDRAINED TRIAXIAL CO



EMBANKMENT FOUNDATION MATERIALS

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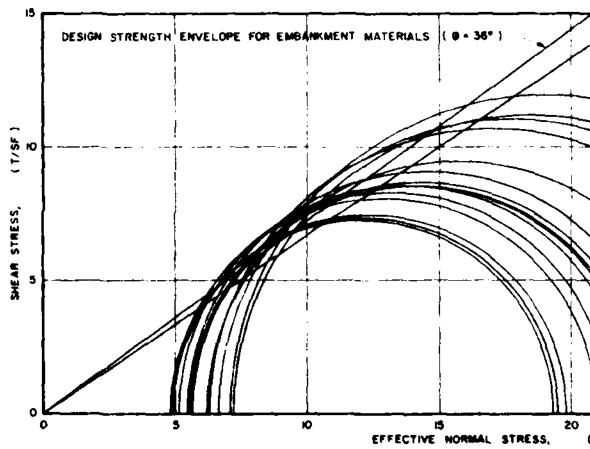
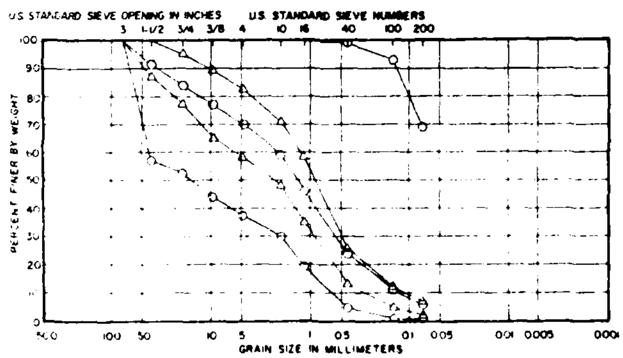
△ — UPPER AND LOWER QUARTILES
○ — MEAN GRADATION

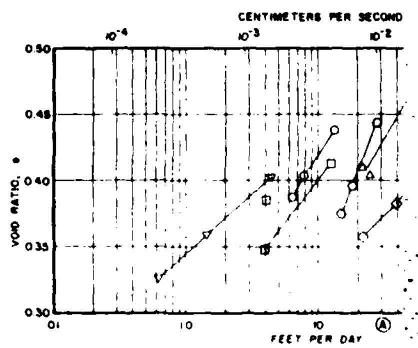


CONSOLIDATED-DRAINED TRIAXIAL CO



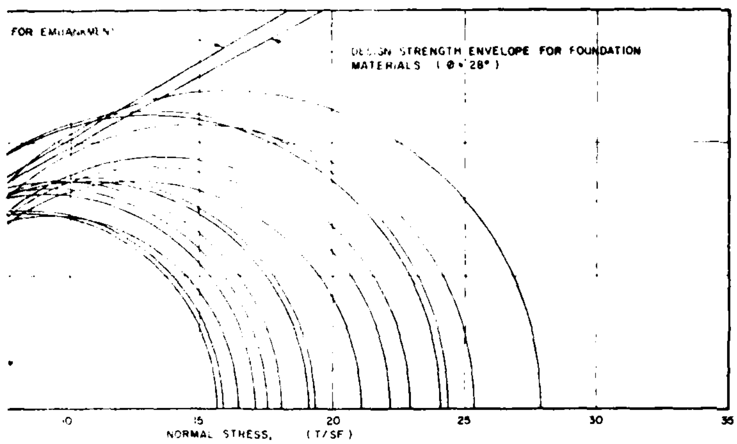
CHANNEL FOUNDATION MATERIALS

○ — UPPER AND LOWER GRADATION LIMITS
△ — UPPER AND LOWER QUARTILES
○ — MEAN GRADATION

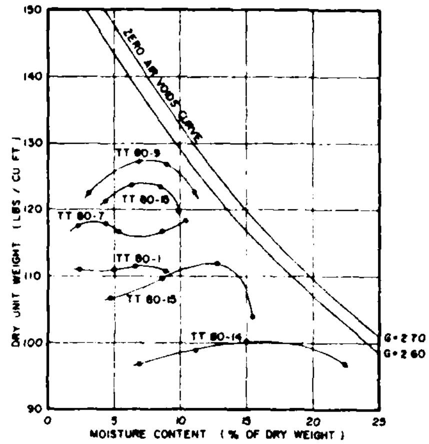


PERMEABILITY

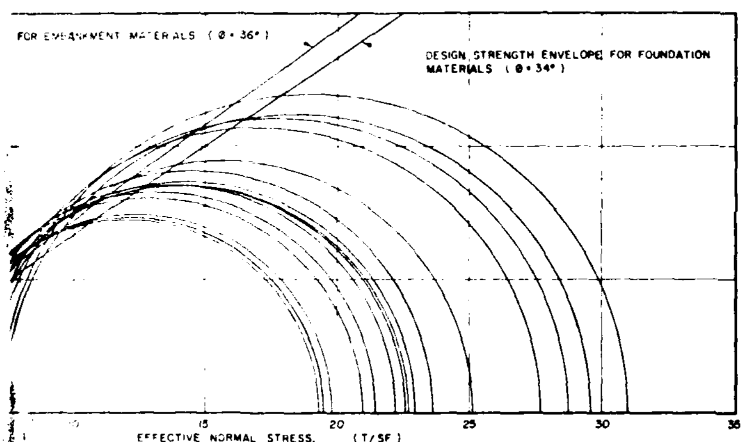
Ⓐ DESIGN PERMEABILITY FOR EMBANKMENT
Ⓑ DESIGN PERMEABILITY FOR FOUNDATION



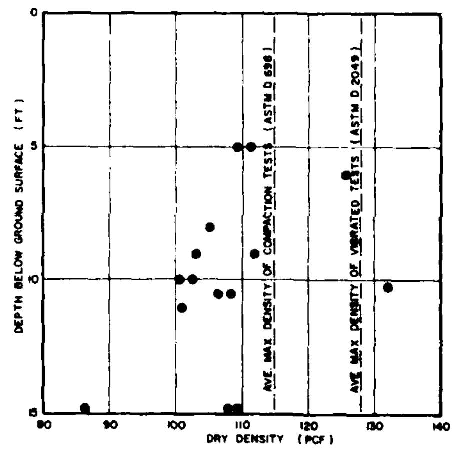
CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS



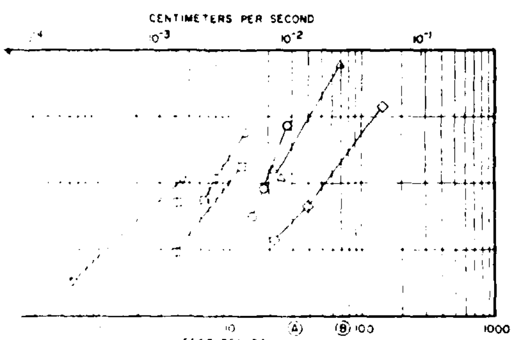
COMPACTION TESTS



CONSOLIDATED-DRAINED TRIAXIAL COMPRESSION TESTS



IN-SITU DENSITY TESTS



PERMEABILITY

(A) DESIGN PERMEABILITY FOR EMBANKMENT MATERIALS (30 FPD)
(B) DESIGN PERMEABILITY FOR FOUNDATION MATERIALS (70 FPD)

SYMBOL | DESCRIPTION | DATE | APPRO
REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: BR
DRAWN BY: BR
CHECKED BY:

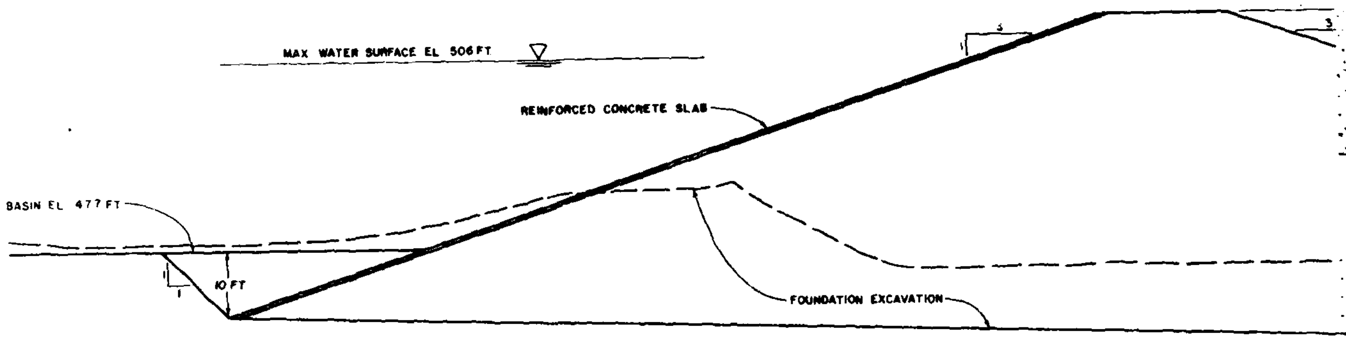
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

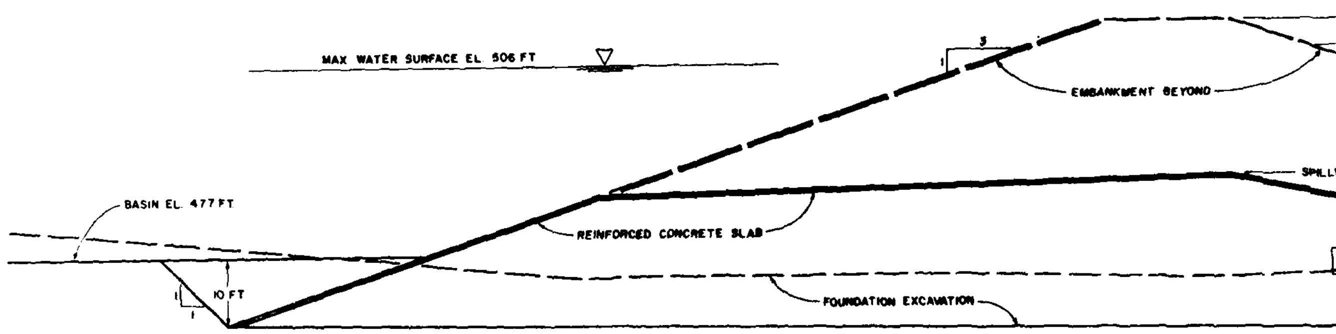
MATERIAL TEST RESULTS

SUBMITTED BY: | DATE APPROVED: | SPEC. NO. DACW09-____-B-____ | SHEET
DISTRICT FILE NO.

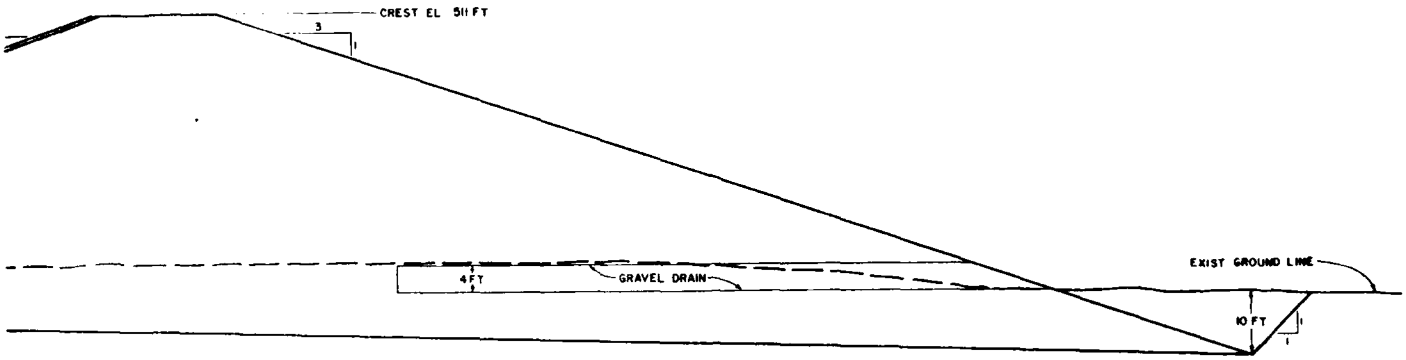
II



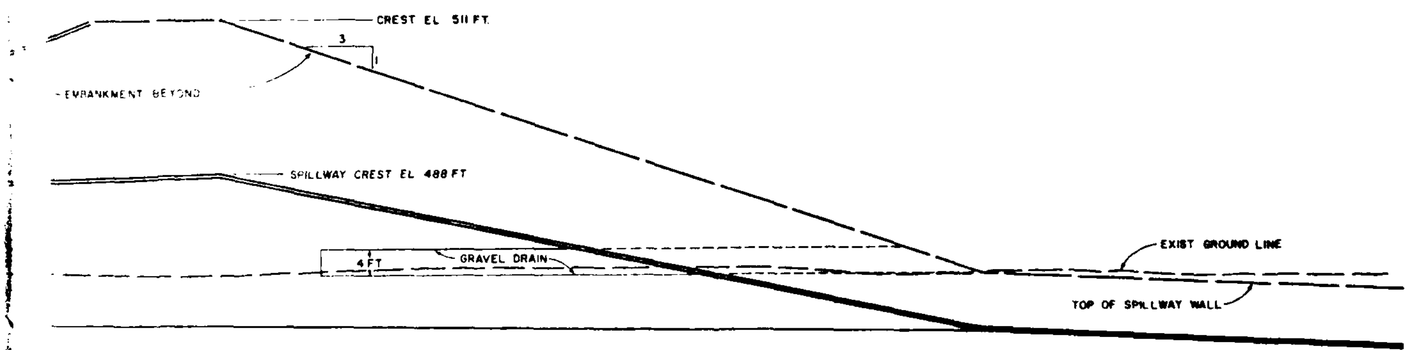
TYPICAL EMBANKMENT SECTION OUTSIDE OF SPIL



EMBANKMENT SECTION AT CENTERLINE OF SPILL



ANKMENT SECTION OUTSIDE OF SPILLWAY



ION AT CENTERLINE OF SPILLWAY

SCALE 10' 0 10' 20' 30' 40'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: RR
DRAWN BY: RR
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

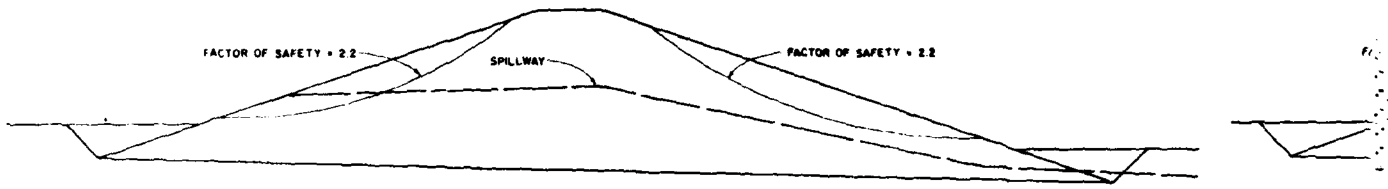
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

EMBANKMENT SECTIONS

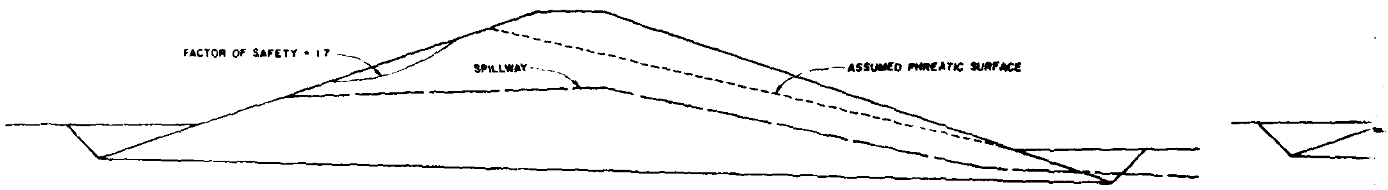
SUBMITTED BY: DATE APPROVED: SPEC NO. DACW09-___-B-___ SHEET

DISTRICT FILE NO.

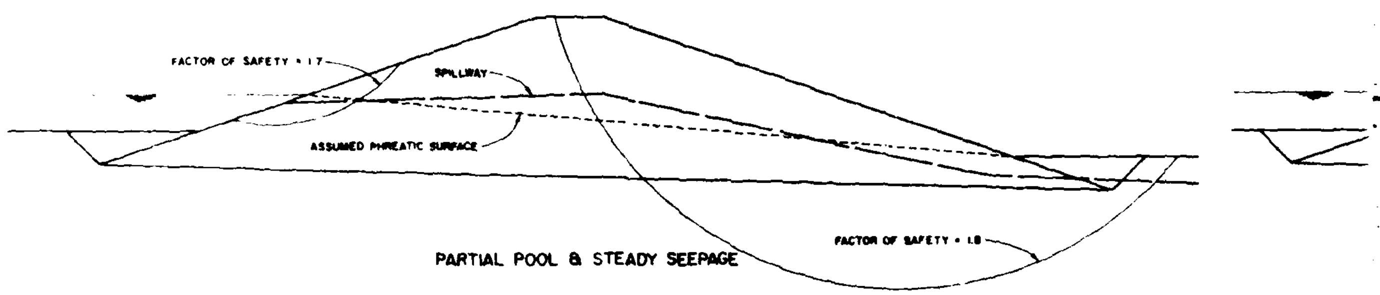
II



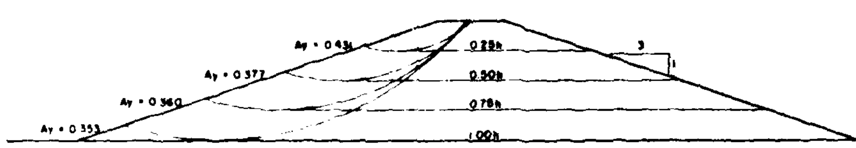
END OF CONSTRUCTION



DRAWDOWN FROM MAXIMUM WATER SURFACE ELEVATION

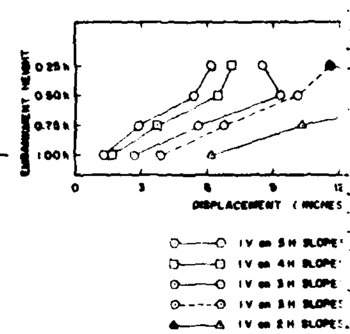


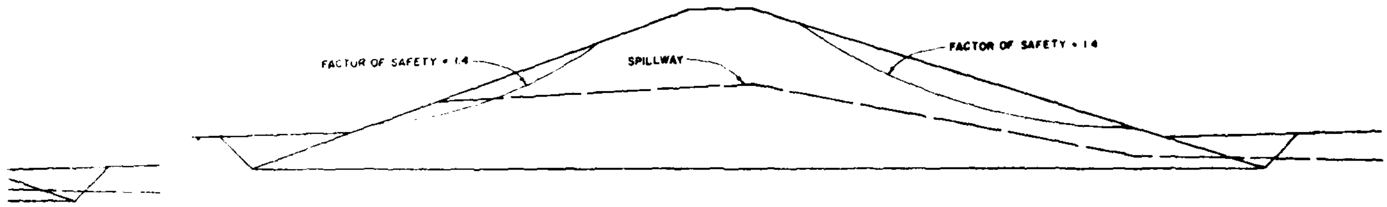
PARTIAL POOL & STEADY SEEPAGE



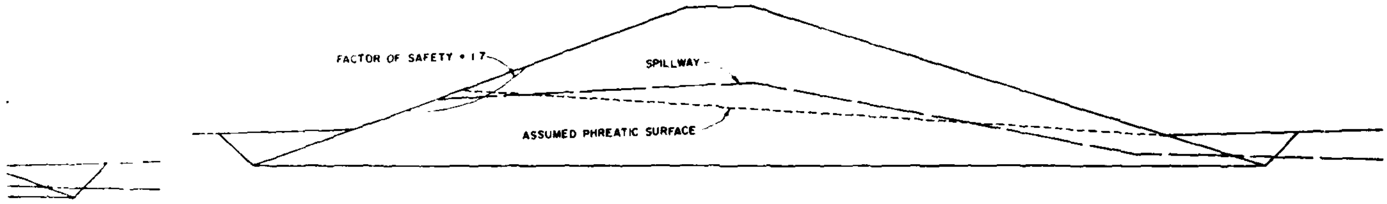
YIELD ACCELERATIONS FOR ASSUMED FAILURE ARCS
(1V on 3H SLOPES SHOWN)

SEISMIC INDUCED SLOPE DISPLACEMENT MODEL

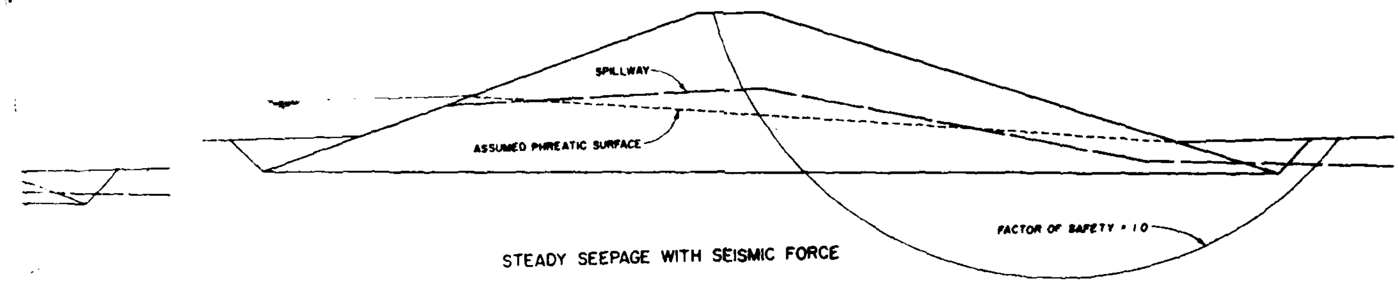




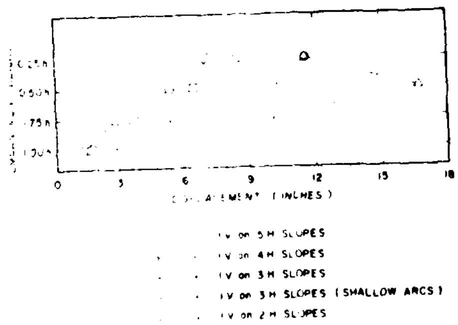
END OF CONSTRUCTION WITH SEISMIC FORCE

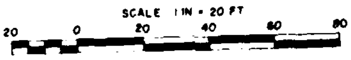


DRAWDOWN FROM SPILLWAY CREST ELEVATION



STEADY SEEPAGE WITH SEISMIC FORCE





SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: E.K.
DRAWN BY: B.R.
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

CIRCULAR ARC
STABILITY AND DISPLACEMENT ANALYSIS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____ B-____
SHEET
DISTRICT FILE NO.

PLATE C-17

II

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY

REPORT
OF
SOIL TESTS

RANCHO MIRAGE DAM

MARCH 1981

SAUSALITO, CALIFORNIA

REPORT
OF
SOIL TESTS

RANCHO MIRAGE DAM

MARCH 1981

AUTHORIZATION

1. Results of tests reported herein were requested by the Los Angeles District in laboratory request No. CIV-81-28 dated 15 December 1980.

SAMPLES

2. Ten disturbed sack samples were received on 13 November 1980. Identification of tested samples are shown on Soil Test Result Summary, plate 1.

TESTING PROGRAM

3. The program was in general accordance with the test request. Tests included sieve analysis, specific gravity, permeability, consolidation and triaxial compression.

TEST METHODS

4. a, Grain-size Analysis, Specific Gravity, Triaxial Compression, Permeability, and Consolidation . Testing methods conformed to the procedure described in Engineer Manual, EM 1110-2-1906, "Laboratory Soil Testing," 30 November 1970.

b. Classification. The soil was classified in accordance with "The Unified Soil Classification System," TM No. 3-357, Appendix A, April 1960.

TEST RESULTS

5. Results of tests are shown on the following plates:

SUBJECT	PLATE NO.
Soil Test Result Summary	1
Triaxial Compression Test Report	2 - 11
Consolidation Test Report	12 - 19
Permeability	20 - 21

CORPS OF ENGINEERS

U.S. ARMY ENGINEER DIVISION

SOIL TEST RESU

PROJECT: RANCHO MIRAGE DAM

DIVISION SERIAL NO.	HOLE NO.	CO-ORD. OR STA.	FIELD SAMPLE NO.	DEPTH OR ELEVATION FROM	DEPTH OR ELEVATION TO	LABORATORY DESCRIPTIVE CLASSIFICATION	MECHANICAL ANALYSIS - % GRAVEL 3 / 2	GRAVEL 1½ / 1	GRAVEL ¾ / ½	GRAVEL ⅜	SAND NO. 4	SAND NO. 10	SAND NO. 40
74121	TP80-1						98 / 96	95 / 93	89 / 86	84	77	53	9
74122	TP80-2						100 / 99	98 / 98	96 / 95	93	87	67	15
74123	TP80-3						100 / 96	94 / 98	84 / 76	70	57	42	23
74124	TP80-4						94 / 91	90 / 89	88 / 86	84	80	64	18
74125	TP80-4A						100 / 99	97 / 94	92 / 90	87	81	67	22
74128	TT80-2						100	99 / 98	97 / 96	95	91	77	30
74133	TT80-3		6	0	15		100 / 96	95 / 92	89 / 83	81	74	58	12
74129	TT80-4						95 / 94	92 / 91	90 / 89	88	84	75	37
74126	TT80-5						100	98 / 96	95 / 93	92	87	66	17
74127	TT80-6						95 / 95	93 / 92	90 / 89	87	83	66	27

U.S. ARMY

VISION LABORATORY —— SOUTH PACIFIC

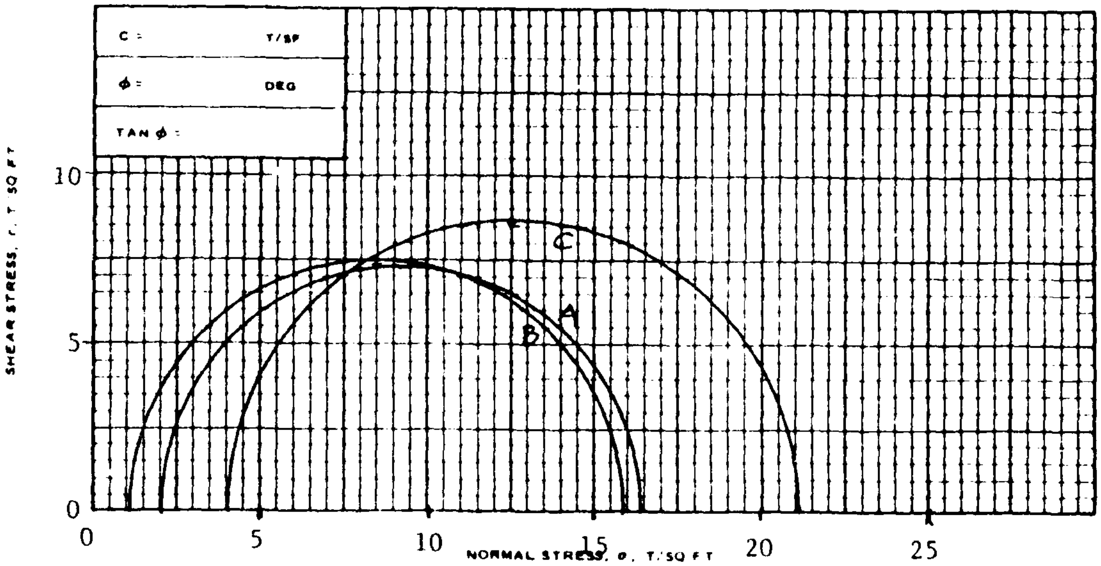
T RESULT SUMMARY

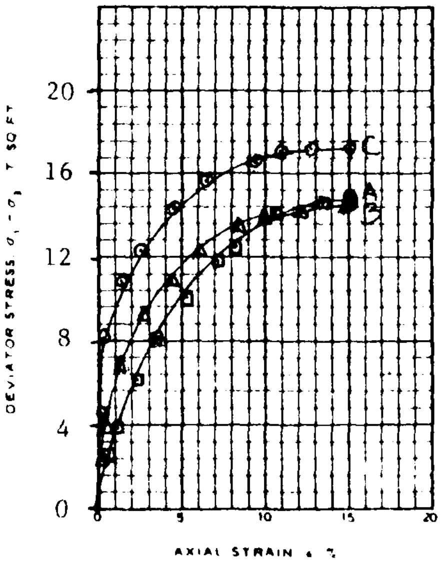
DATE: February 1981

SIS - % FINER					LIQUID LIMIT	PLASTICITY INDEX	SPECIFIC GRAVITY		FIELD		RESULTS ON SEPARATE FORMS						
SAND			FINES								COMPACTION		U=UNDISTURBED			R=REMOLDED	
											AASHO.		SHEAR STRENGTH		CONSOLIDATION	PERMEABILITY	
10	NO 40	NO 60	NO 200				+4	-4	DRY WT LB/CF	% MOIST	LB/CF	% MOIST.	DIRECT	TRIAXIAL			
53	[illegible]	5	3				2.73	2.71						3-R		3-R	
57	15	9	3					2.71						3-R	1-R		
42	23	18	7					2.76								3-R	
4	18	10	3					2.71								3-R	
7	22	13	4					2.72								3-R	
7	30	19	7					2.72								3-R	
8	12	7	2				2.74	2.73						3-R			
5	37	25	9					2.74						3-R	1-R	3-R	
6	17	9	3					2.72								3-R	
[illegible]	27	16	5				2.75	2.70						3-R			

PLATE I

II





SPECIMEN NO			A	B	C
INITIAL	WATER CONTENT, %	w_o	0.6	0.6	0.7
	DRY DENSITY LB/CU FT	γ_{d_o}	120.3	120.7	120.1
	SATURATION, %	S_o	4	4	5
	VOID RATIO	e_o	0.410	0.406	0.414
BEFORE SHEAR	WATER CONTENT, %	w_c	14.3	14.6	14.5
	DRY DENSITY LB/CU FT	γ_{d_c}	121.6	121.8	121.8
	SATURATION, %	S_c	98	100	100
	VOID RATIO	e_c	0.396	0.393	0.393
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.48	6.48	6.48
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	14.93	14.46	17.11
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	270	248	249
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	–	–	–
INITIAL DIAMETER, IN		D_o	6.0	6.0	6.0
INITIAL HEIGHT, IN		H_o	13.75	13.73	13.77

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Gravelly Sand (SP)

LL	PL	PI	Gs 2.72	TYPE OF SPECIMEN	TYPE OF TEST $\overline{R}$

REMARKS

PROJECT RANCHO MIRAGE DAM

BORING NO TP80-1 SAMPLE NO 74121

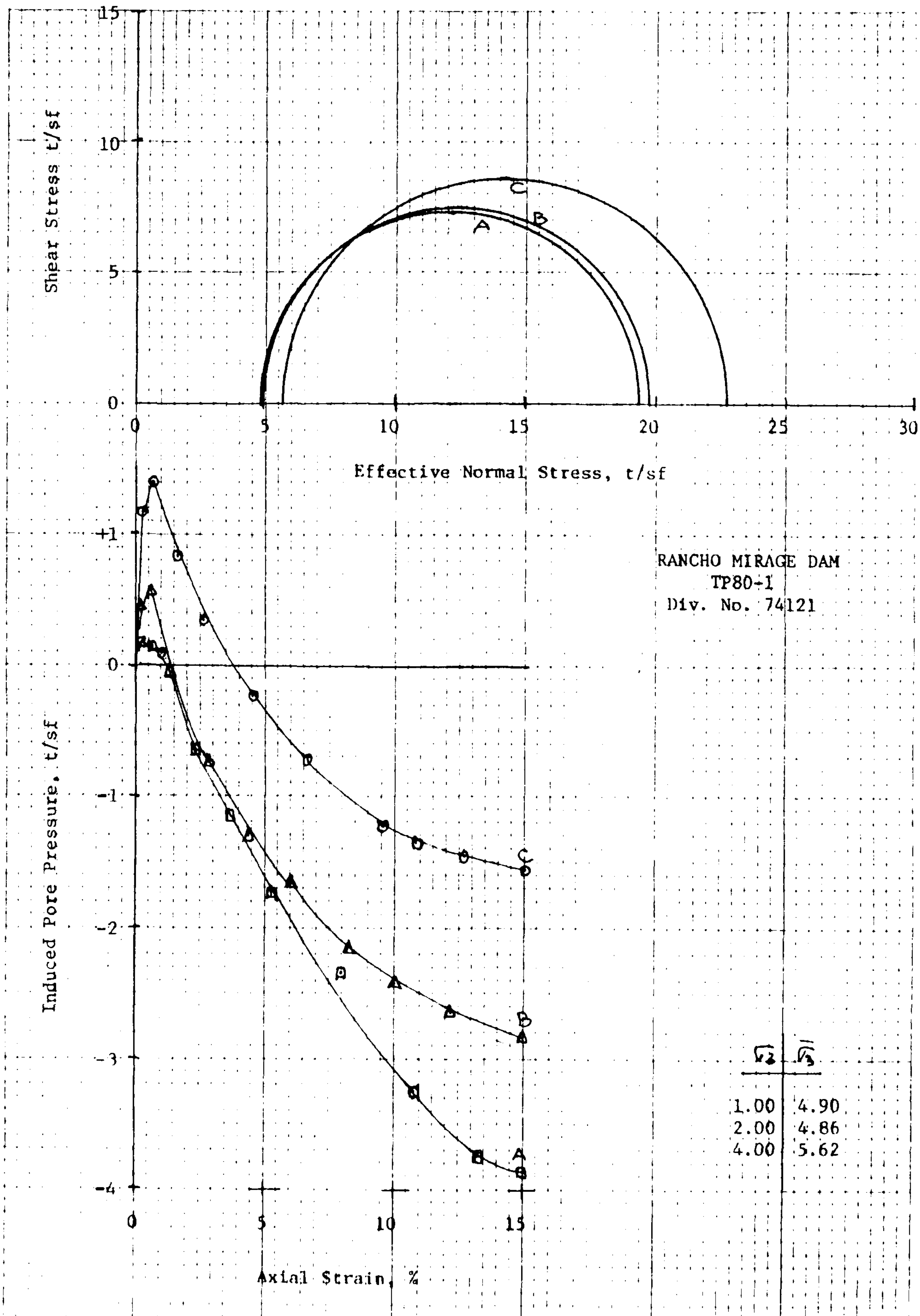
DEPTH ELEV

LABORATORY SPDL DATE February 1981

TRIAXIAL COMPRESSION TEST REPORT

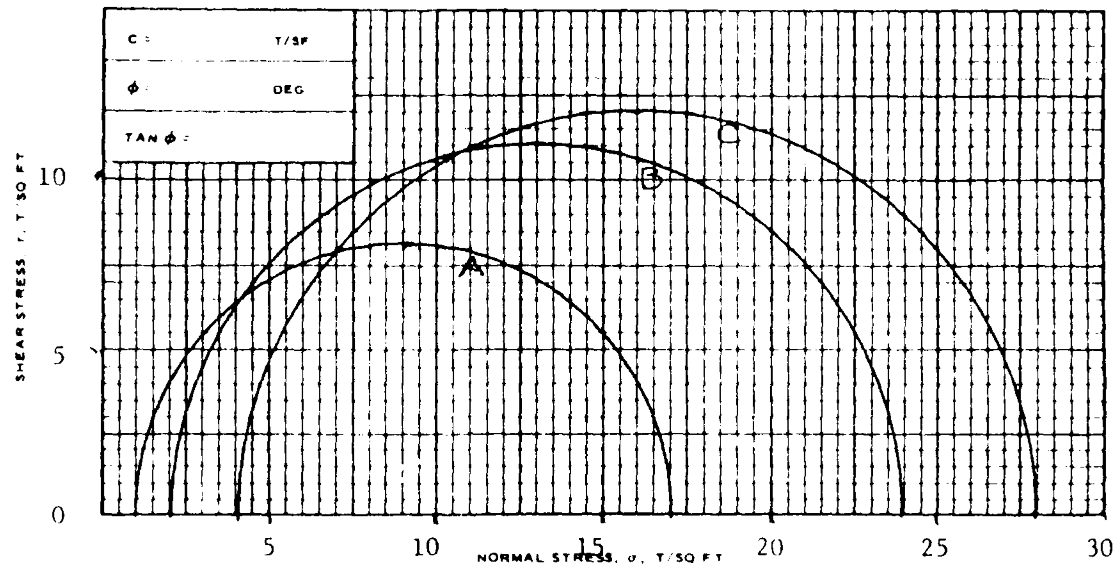
ENG FORM NO 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

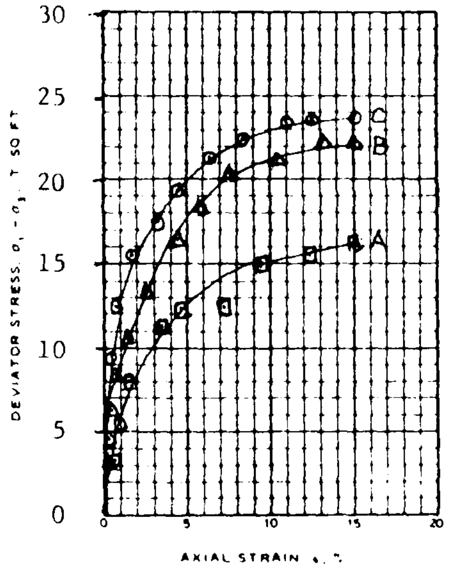
Plate 2



Shear Stress t/sf
Effective Normal Stress, t/sf
A
B
C
Induced Pore Pressure, t/sf
Axial Strain, %
RANCHO MIRAGE DAM
TP80-1
Div. No. 74121
1.00 4.90
2.00 4.86
4.00 5.62

Plate 3





SPECIMEN NO			A	B	C	
INITIAL	WATER CONTENT, %	w_o	0.6	0.6	0.6	
	DRY DENSITY LB/CU FT	γ_{d_o}	121.1	121.9	121.7	
	SATURATION, %	S_o	4	4	4	
	VOID RATIO	e_o	0.396	0.387	0.390	
BEFORE SHEAR	WATER CONTENT, %	w_c	14.4	13.6	12.9	
	DRY DENSITY LB/CU FT	γ_{d_c}	121.8	123.7	122.8	
	SATURATION, %	S_c	100	100	99	
	VOID RATIO	e_c	0.389	0.367	0.377	
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.48	6.48	6.48	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	16.13	22.13	23.90	
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	250	270	282	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	–	–	–	
INITIAL DIAMETER, IN		D_o	6.0	6.0	6.0	
INITIAL HEIGHT, IN		H_o	13.76	13.75	13.74	

CONTROLLED-STRAIN TEST

DESCRIPTION OF SPECIMENS: Sand (SP)

LL | PL | PI | G_s 2.71 | TYPE OF SPECIMEN: Remolded | TYPE OF TEST: $\bar{R}$

REMARKS

PROJECT: RANCHO MIRAGE DAM

BORING NO: TP80-2 | SAMPLE NO: 74122

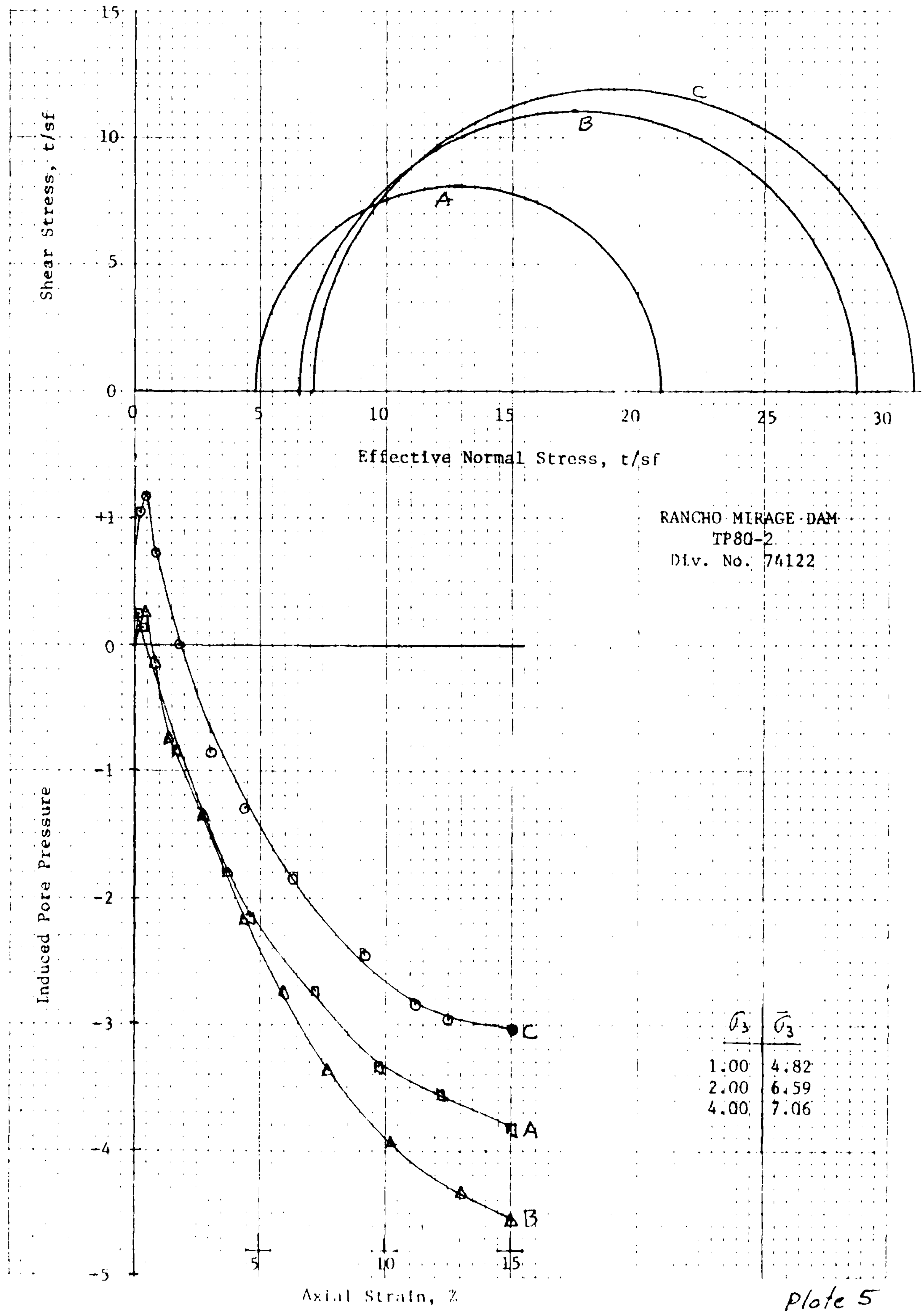
DEPTH ELEV

LABORATORY: SPDL | DATE: February 1981

TRIAXIAL COMPRESSION TEST REPORT

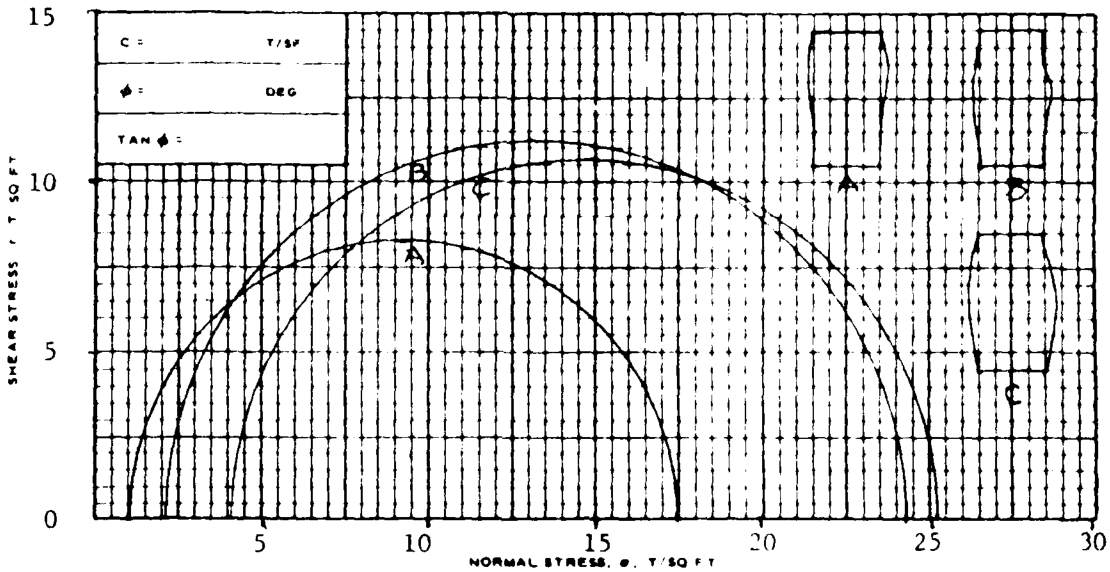
ENG FORM NO 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

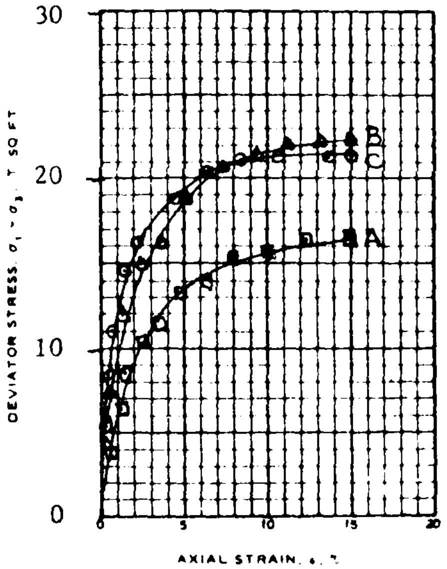
Plate 4



Shear Stress, t/sf
15
10
5
0
0
5
10
15
20
25
30
Effective Normal Stress, t/sf
A
B
C
RANCHO MIRAGE DAM
TP80-2
Div. No. 74122
Induced Pore Pressure
+1
0
-1
-2
-3
-4
-5
5
10
15
Axial Strain, %
C
A
B
σ_3 | $\bar{\sigma}_3$
1.00 | 4.82
2.00 | 6.59
4.00 | 7.06

Plate 5





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	0.7	0.3	0.5
	DRY DENSITY LB/CU FT	γ_{d_o}	118.3	118.9	118.7
	SATURATION, %	S_o	4	2	3
	VOID RATIO	e_o	0.440	0.433	0.435
BEFORE SHEAR	WATER CONTENT, %	w_c	15.8	15.1	15.0
	DRY DENSITY LB/CU FT	γ_{d_c}	119.8	120.3	120.2
	SATURATION, %	S_c	98	99	98
	VOID RATIO	e_c	0.422	0.416	0.417
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.48	6.48	6.48
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1-\sigma_3)_{max}$	16.51	22.41	21.39
TIME TO $(\sigma_1-\sigma_3)_{max}$, MIN		t_f	240	253	220
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1-\sigma_3)_{ult}$	–	–	–
INITIAL DIAMETER, IN		D_o	6.0	6.0	6.0
INITIAL HEIGHT, IN		H_o	13.76	13.76	13.77

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Gravelly Sand

LL | PL | PI | G_s 2.73 | TYPE OF SPECIMEN Remolded | TYPE OF TEST R

REMARKS

PROJECT RANCHO MIRAGE DAM

BORING NO. TT-80-3 SAMPLE NO. 74133

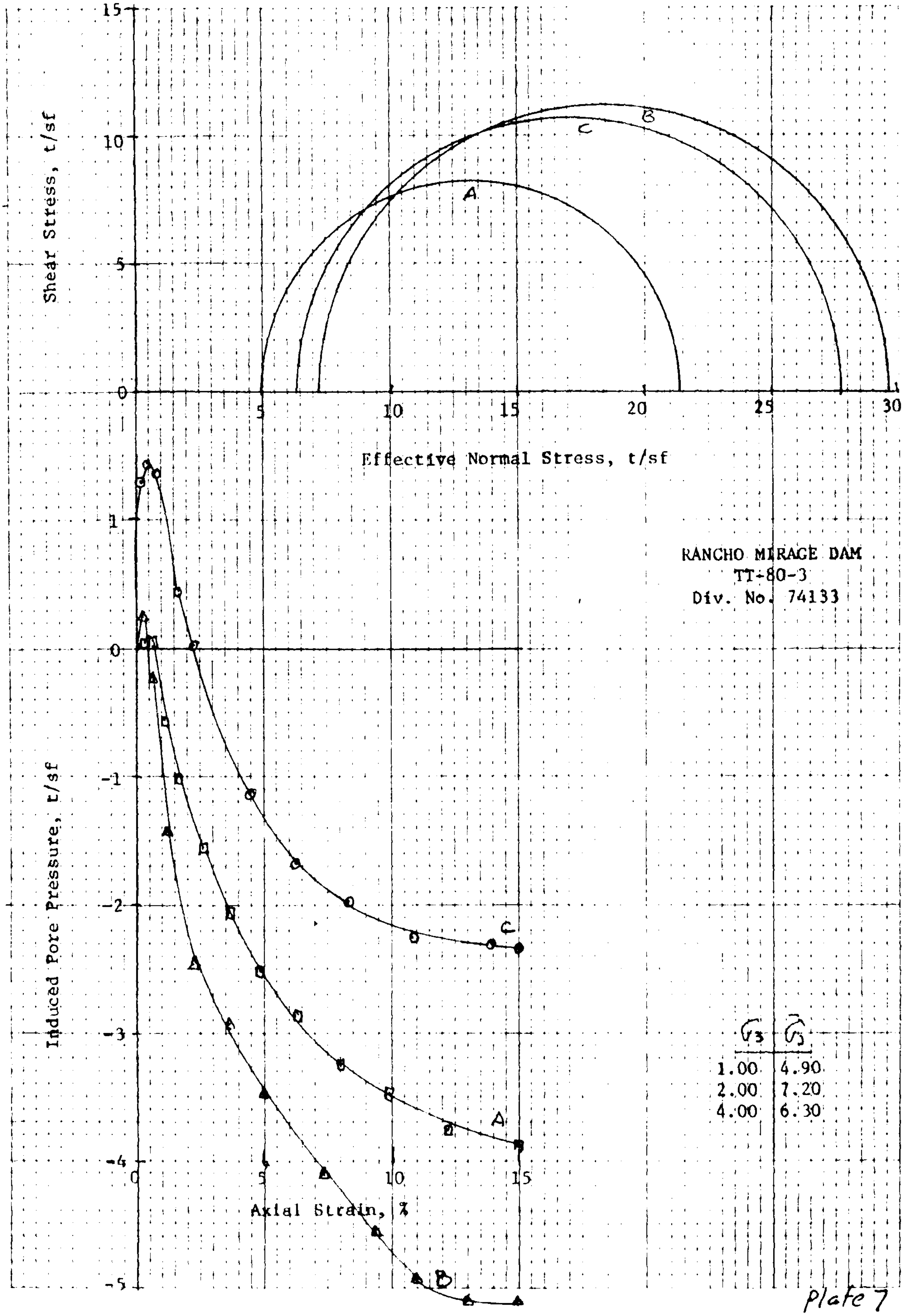
DEPTH ELEV 0 - 15'

LABORATORY SPDL DATE February 1981

TRIAXIAL COMPRESSION TEST REPORT

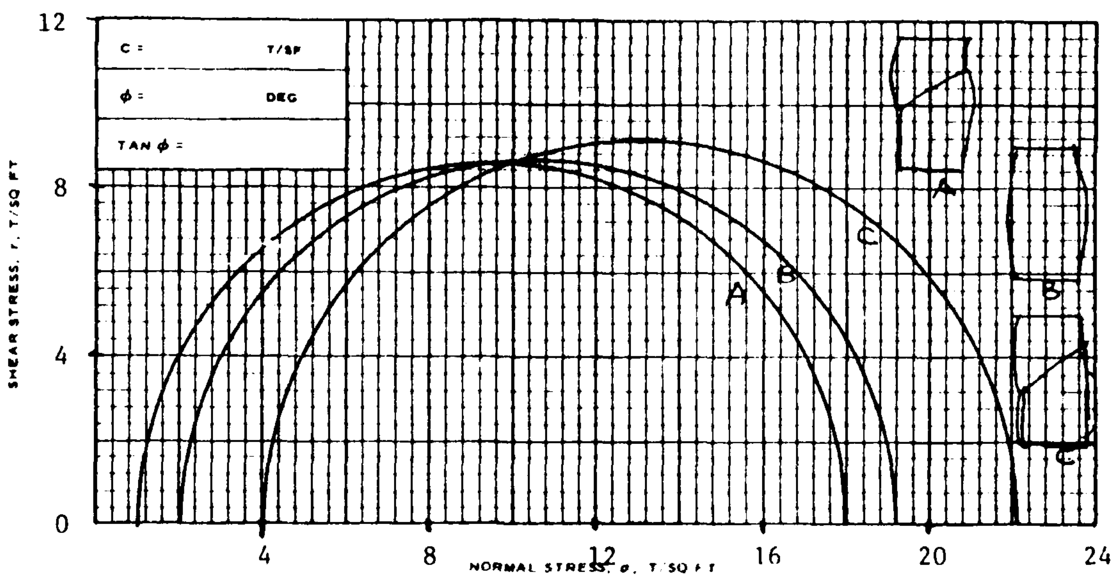
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE. TRANSLUCENT (EM 1110-2-1906)

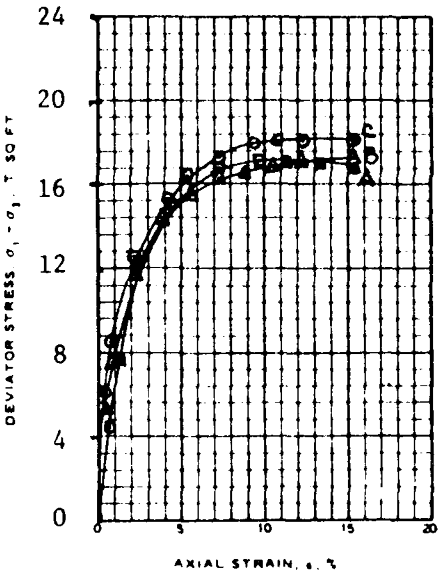
plate 6



Shear Stress, t/sf
Effective Normal Stress, t/sf
Induced Pore Pressure, t/sf
Axial Strain, %
A
B
C
RANCHO MIRAGE DAM
TT-80-3
Div. No. 74133
σ3 σ̄3
1.00 4.90
2.00 7.20
4.00 6.30

Plate 7





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	0	0	0
	DRY DENSITY LB/CU FT	γd_o	121.6	120.5	120.8
	SATURATION, %	S_o	0	0	0
	VOID RATIO	e_o	0.406	0.418	0.415
BEFORE SHEAR	WATER CONTENT, %	w_c	14.5	14.4	14.3
	DRY DENSITY LB/CU FT	γd_c	121.6	122.0	122.3
	SATURATION, %	S_c	98	98	99
	VOID RATIO	e_c	0.406	0.401	0.398
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.48	6.48	6.48
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	17.07	17.30	18.17
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	207	265	245
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	–	–	–
INITIAL DIAMETER, IN		D_o	6.0	6.0	6.0
INITIAL HEIGHT, IN		H_o	13.76	13.77	13.77

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Gravelly Sand

LL	PL	PI	G 2.74	TYPE OF SPECIMEN Remolded	TYPE OF TEST R

REMARKS

PROJECT RANCHO MIRAGE DAM

BORING NO TT-80-4 | SAMPLE NO 74129

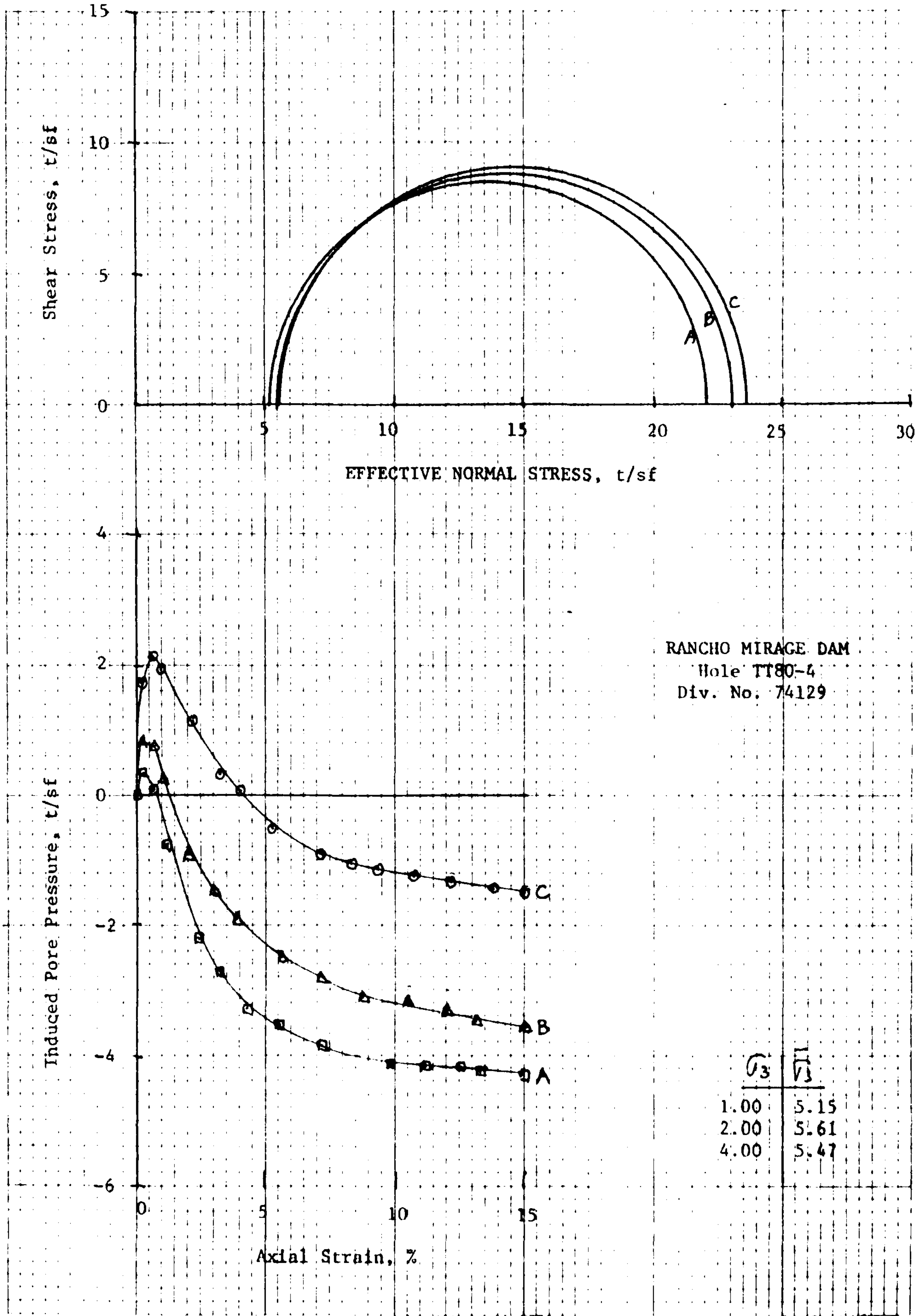
DEPTH ELEV

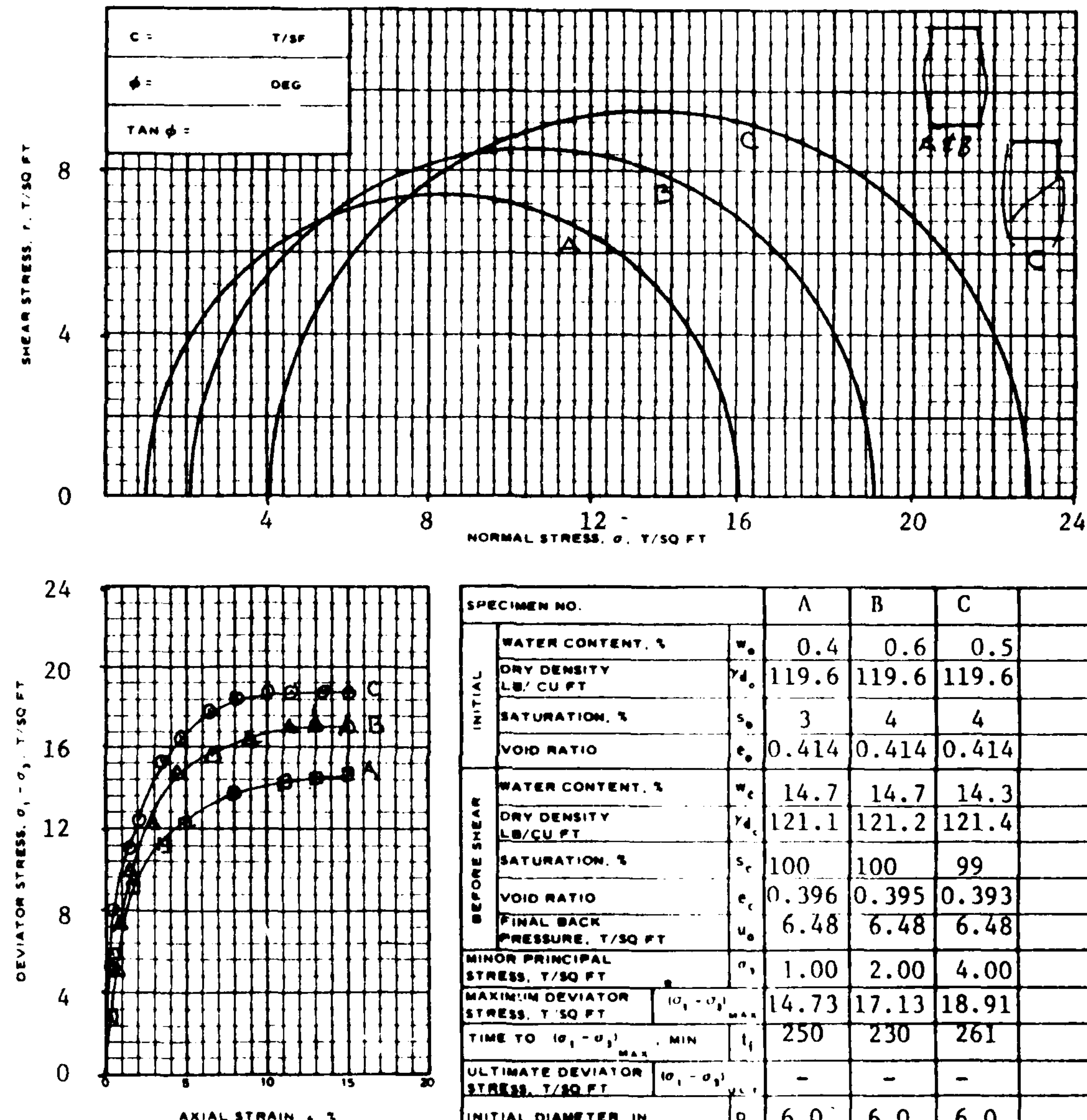
LABORATORY SPDL | DATE March 1981

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

Plate 8


Shear Stress, t/sf
EFFECTIVE NORMAL STRESS, t/sf
A
B
C
Induced Pore Pressure, t/sf
Axial Strain, %
RANCHO MIRAGE DAM
Hole TT80-4
Div. No. 74129
$\bar{\sigma}_3$ $\bar{\sigma}_1/\bar{\sigma}_3$
1.00 5.15
2.00 5.61
4.00 5.47



SPECIMEN NO.			A	B	C	
INITIAL	WATER CONTENT, %	w_o	0.4	0.6	0.5	
	DRY DENSITY LB/CU FT	γd_o	119.6	119.6	119.6	
	SATURATION, %	S_o	3	4	4	
	VOID RATIO	e_o	0.414	0.414	0.414	
BEFORE SHEAR	WATER CONTENT, %	w_c	14.7	14.7	14.3	
	DRY DENSITY LB/CU FT	γd_c	121.1	121.2	121.4	
	SATURATION, %	S_c	100	100	99	
	VOID RATIO	e_c	0.396	0.395	0.393	
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.48	6.48	6.48	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	14.73	17.13	18.91	
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	250	230	261	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	-	-	-	
INITIAL DIAMETER, IN		D_o	6.0	6.0	6.0	
INITIAL HEIGHT, IN		H_o	13.78	13.79	13.79	

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Sand

LL	PL	PI	G: 2.71	TYPE OF SPECIMEN Remolded	TYPE OF TEST R

REMARKS:	PROJECT RANCHO MIRAGE DAM
	BORING NO TT-80-6 / SAMPLE NO. 74127
	DEPTH ELEV
	LABORATORY SPDL / DATE February 1981
	TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)
REV JUNE 1970

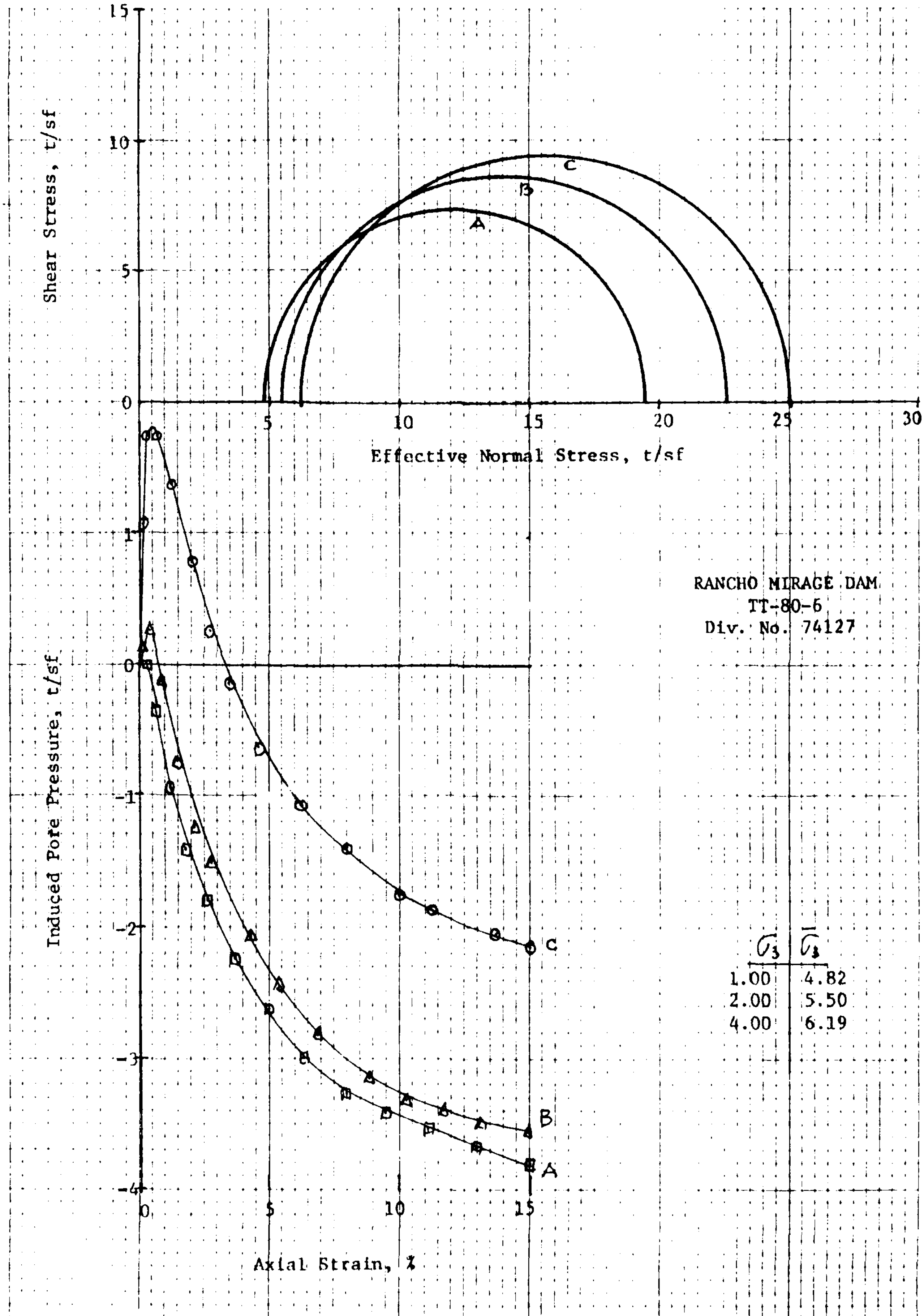
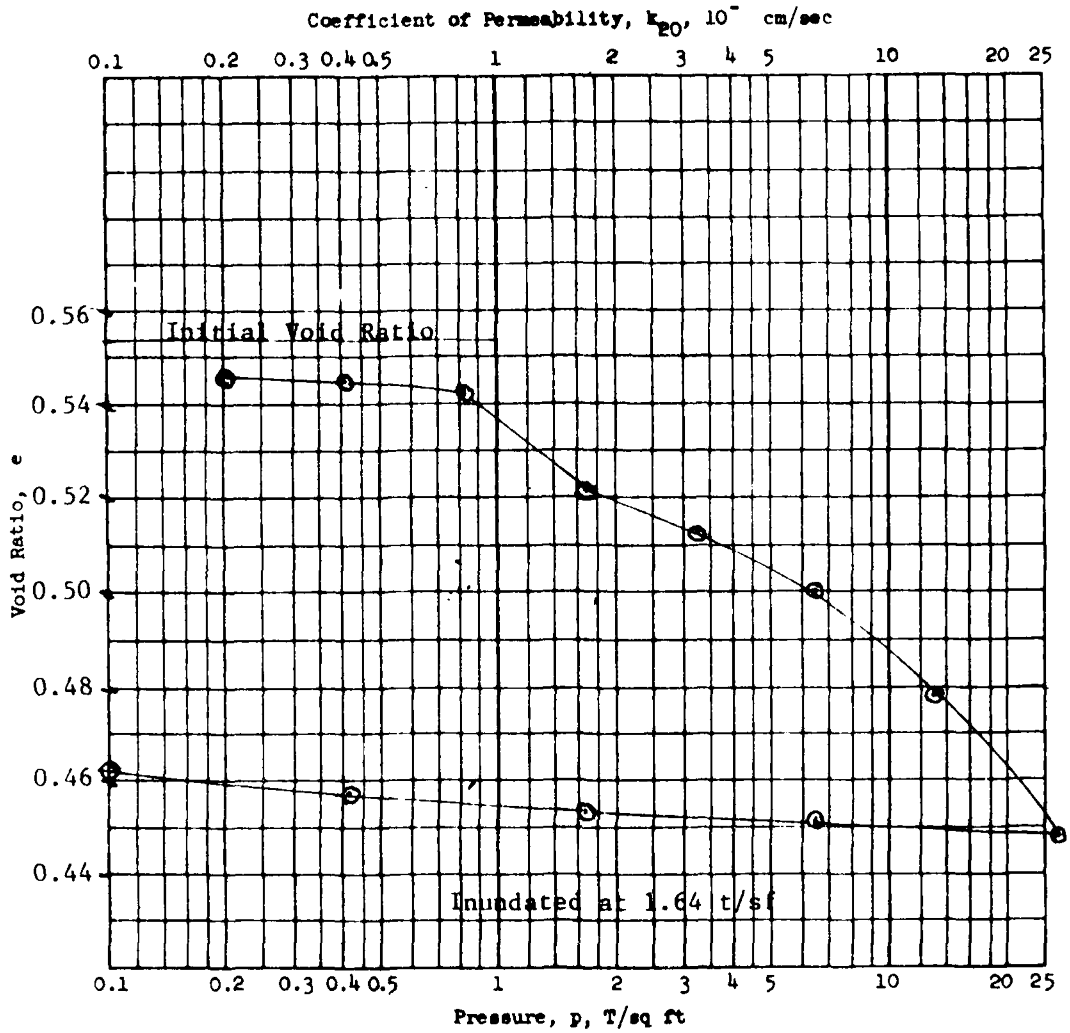

Shear Stress, t/sf
Effective Normal Stress, t/sf
Induced Pore Pressure, t/sf
Axial Strain, %
A
B
C
RANCHO MIRAGE DAM
TT-80-6
Div. No. 74127
σ_3 $\bar{\sigma}_3$
1.00 4.82
2.00 5.50
4.00 6.19

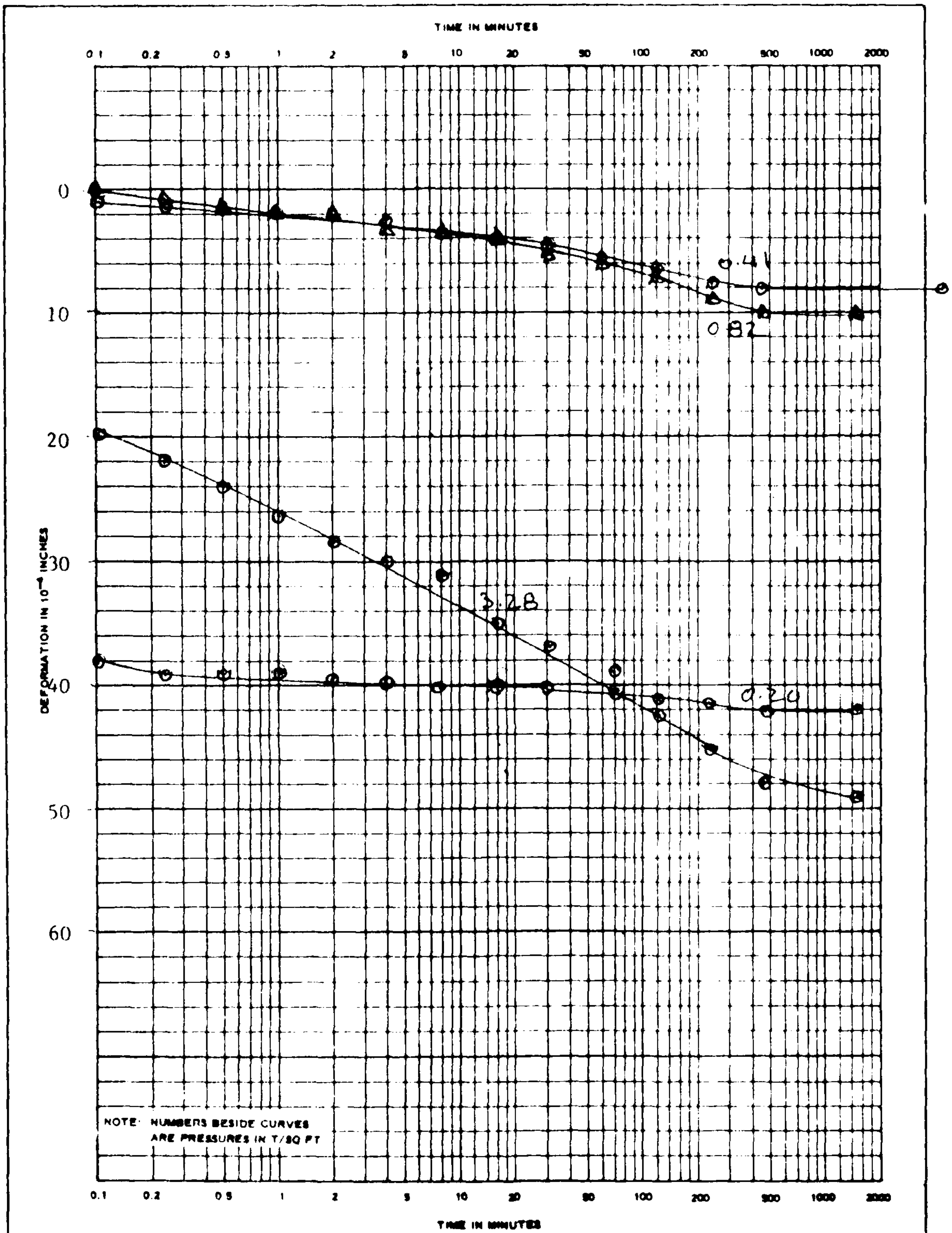
Plate 11



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	0.5 %	w_f	17.3 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.553	e_f	0.469
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	2 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	113.3 lb/ft^3		
Classification		k_{20} at e_o = × 10^{-} cm/sec			
LL	G_s 2.71	Project RANCHO MIRAGE DAM			
PL	D_{10}				
Remarks		Area			
		Boring No. TP80-2		Sample No. 74122	
		Depth El		Date February 1981	
		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) 0 3424

Plate 12



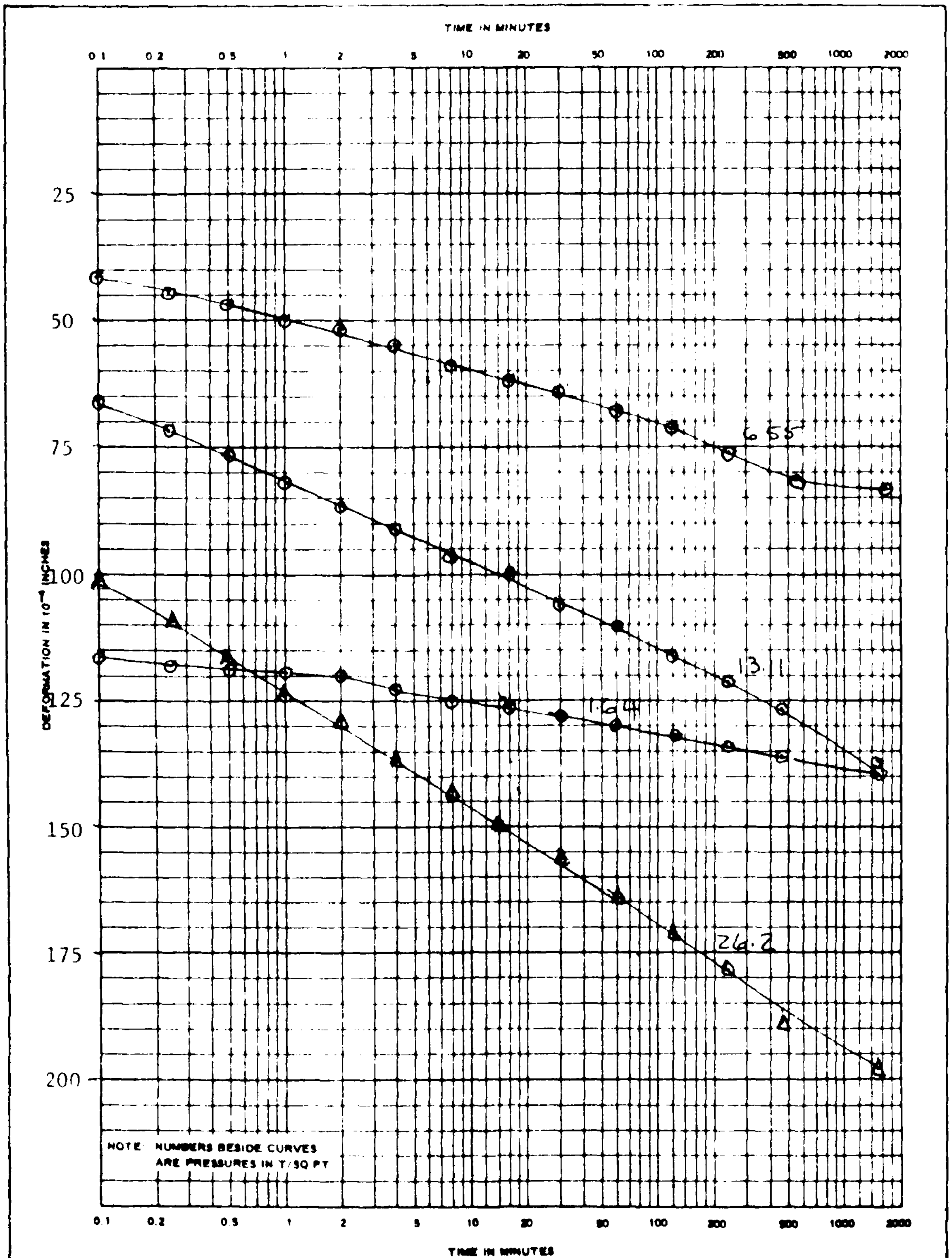
Project RANCHO MIRAGE DAM			
Area			
Boring No. TP80-2	Sample No. 74122	Depth El	Date February 1981

ENG FORM 2088, 1 MAY 63, PREVIOUS EDITIONS ARE OBSOLETE

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

Plate 13



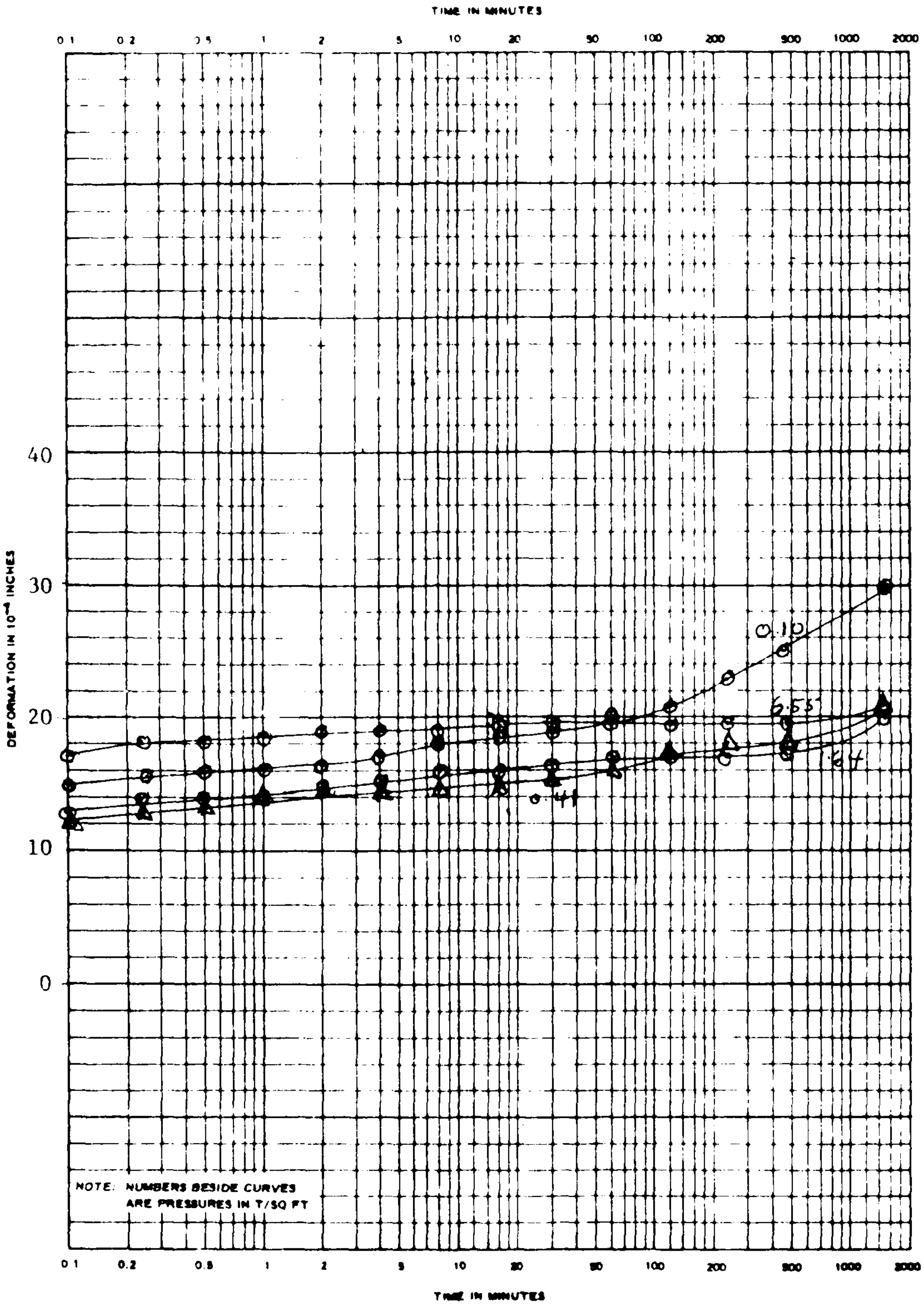
Project Rancho Mirage Dam			
Area			
Boring No. TP80-2	Sample No. 74122	Depth El	Date February 1981

ENG FORM 1 MAY 63 2089 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

Plate 14



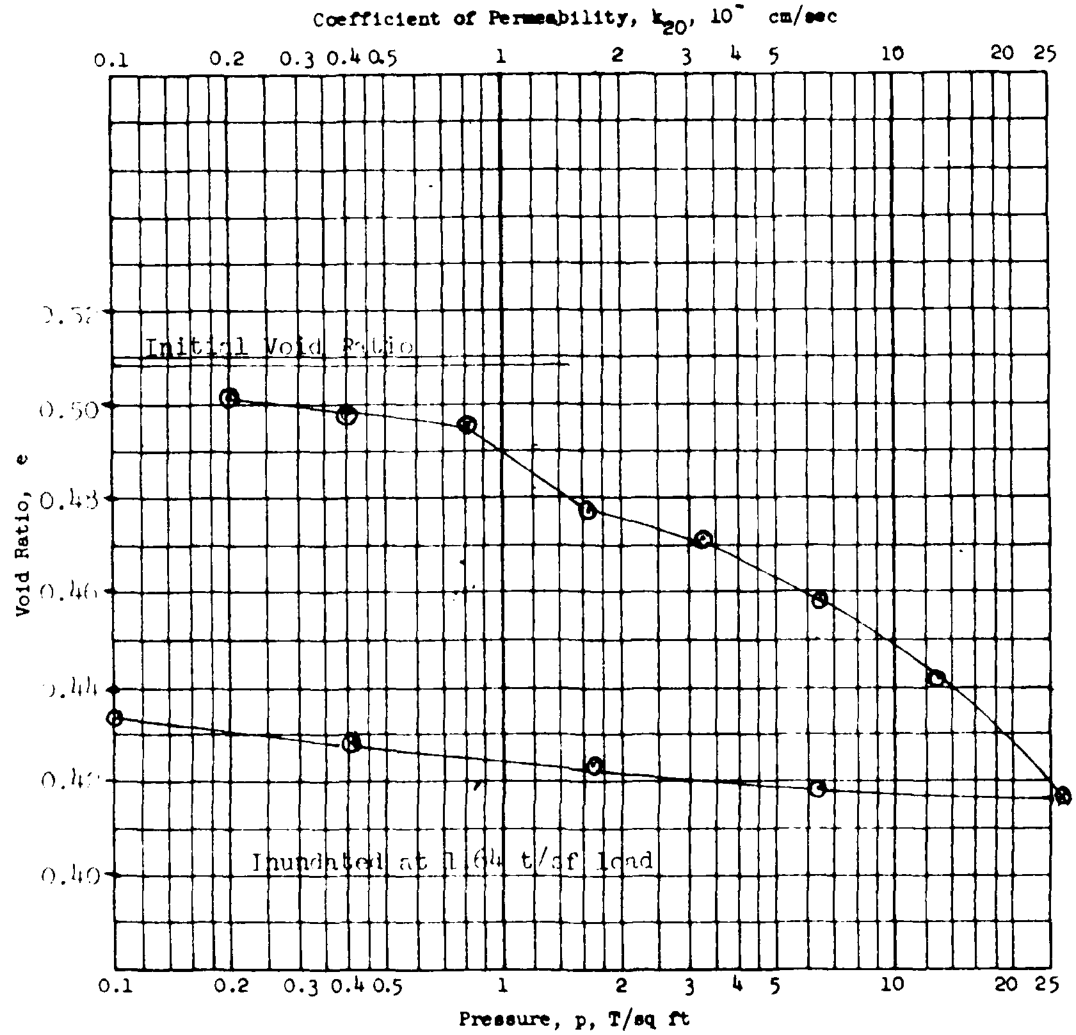
Project	RANCHO MIRAGE DAM		
Area			
Boring No. TP80-2	Sample No. 74122	Depth El	Date February 1981

ENG FORM 2088, 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE

CONSOLIDATION TEST--TIME CURVES

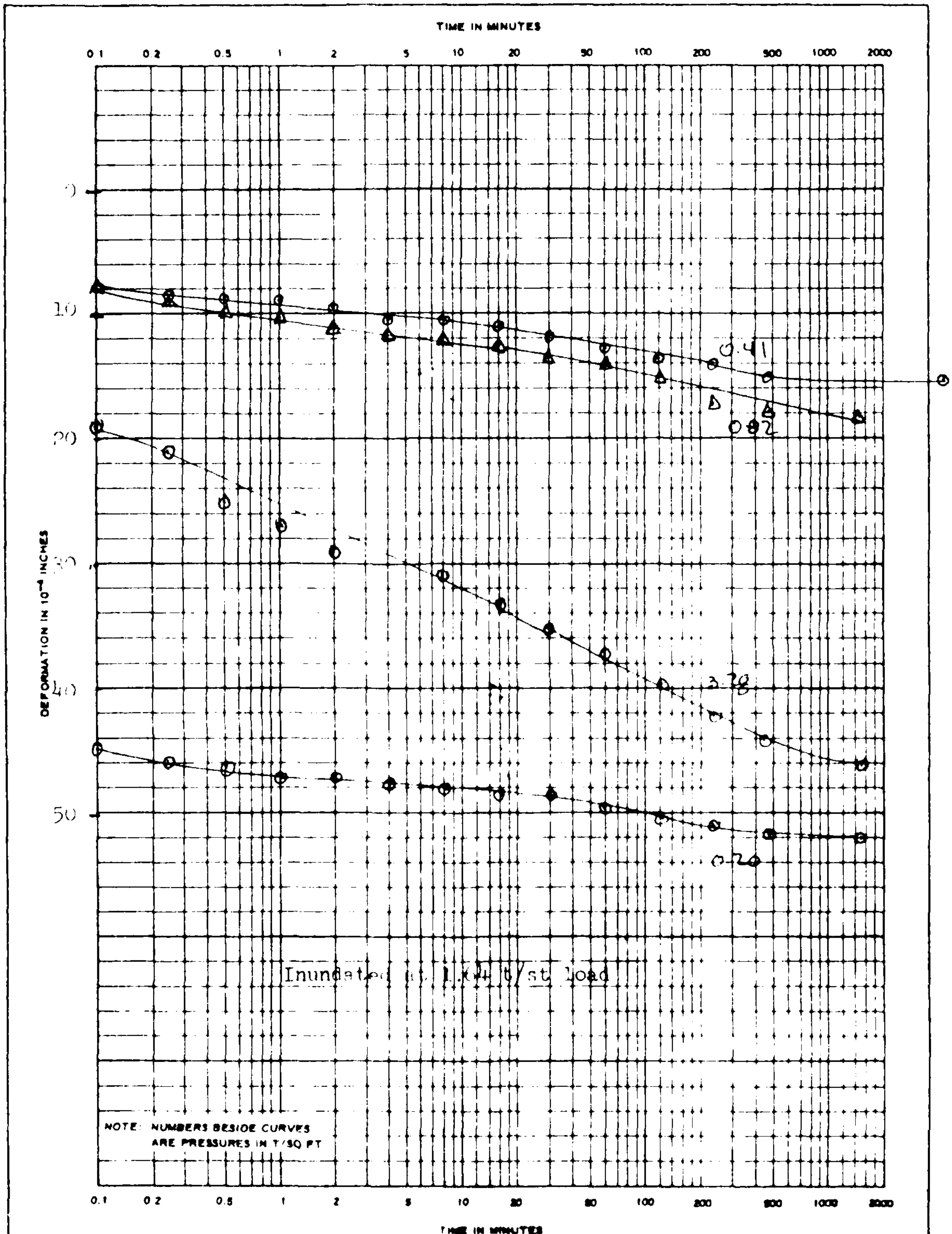
(TRANSLUCENT)

Plate 15



Type of Specimen Remolded		**Before Test**		**After Test**
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	0.5 %	w_f 16.9 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.509	e_f 0.456
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	? %	S_f 100 %
Compression Index, C_c		Dry Density, γ_d	113. lb/ft^3	
Classification Silty Gravelly Sand (SP-SM)		k_{20} at e_o = × 10^{-} cm/sec		
LL	G_s 2.74	Project RANCHO MIRAGE DAM		
PL	D_{10}			
Remarks		Area		
		Boring No. TP 80-4	Sample No. 74199	
		Depth El	Date February 1981	

CONSOLIDATION TEST REPORT

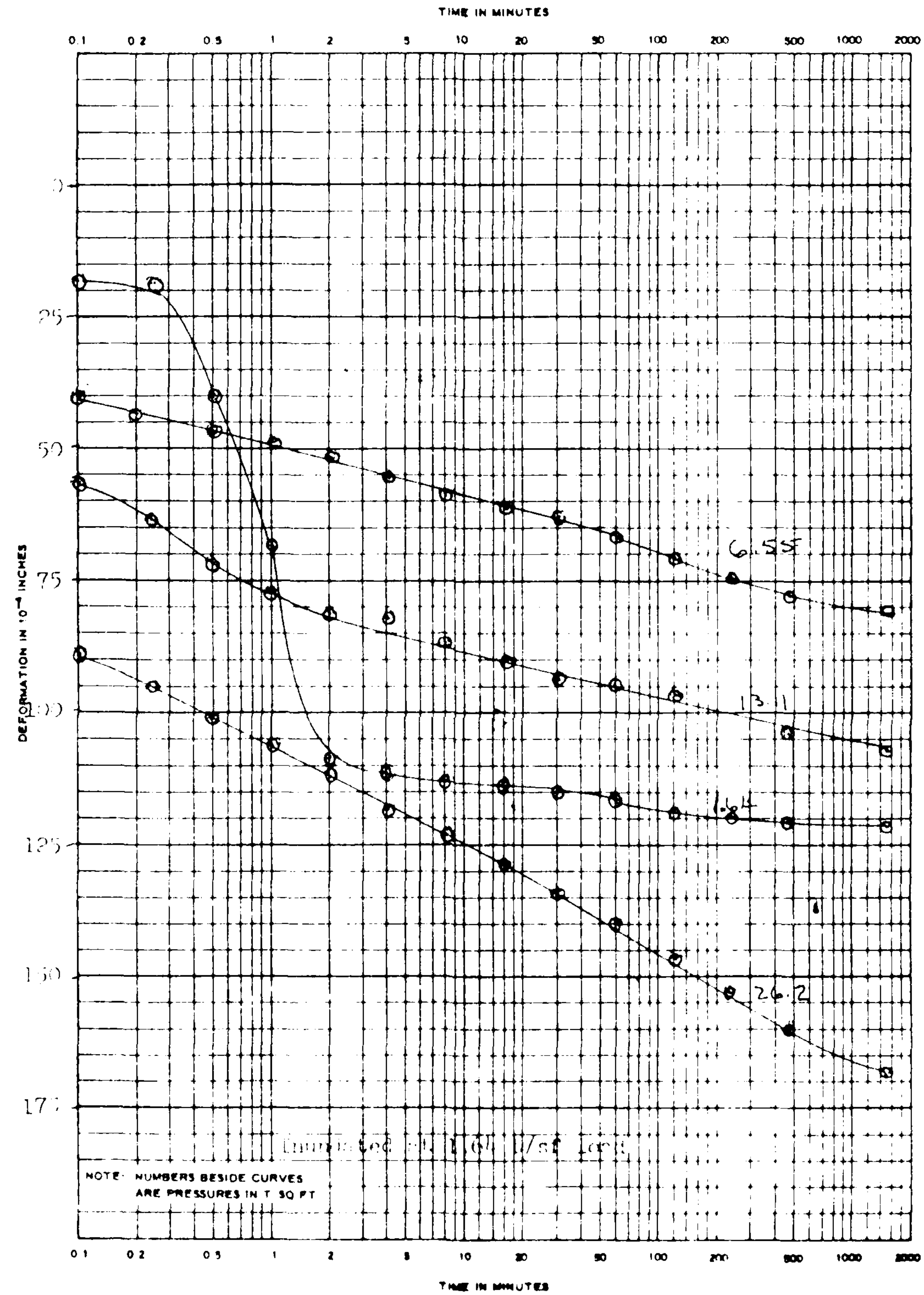


Project RANCHO MIRAGE DAM			
Area			
Boring No. TP 80-4	Sample No. 7412	Depth El	Date February 1981

ENG FORM 2088, 1 MAY 63, PREVIOUS EDITIONS ARE OBSOLETE

CONSOLIDATION TEST--TIME CURVES (TRANSLUCENT)

Plate 17



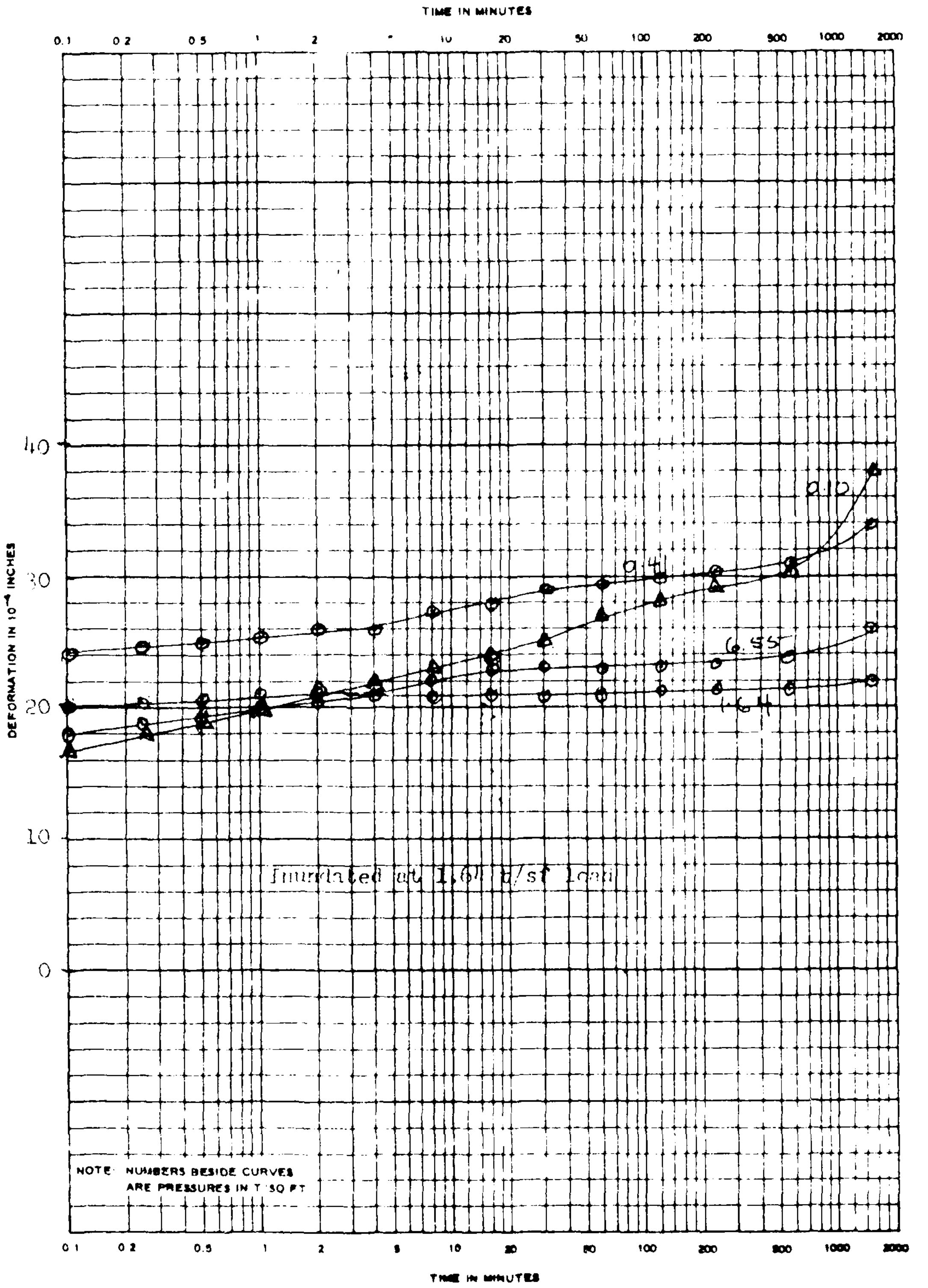
Project RANCHO MIRAGE DAM			
Area			
Boring No. TP 60-[illegible]	Sample No. 7-1[illegible]	Depth El	Date February 1981

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE

CONSOLIDATION TEST--TIME CURVES

TRANSLUCENT

Plate 18



Project [illegible]

Area

Boring No. [illegible]	Sample No. [illegible]	Depth El	Date February 1981

ENG FORM 2088 1 MAY 61 PREVIOUS EDITIONS ARE OBSOLETE

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

p

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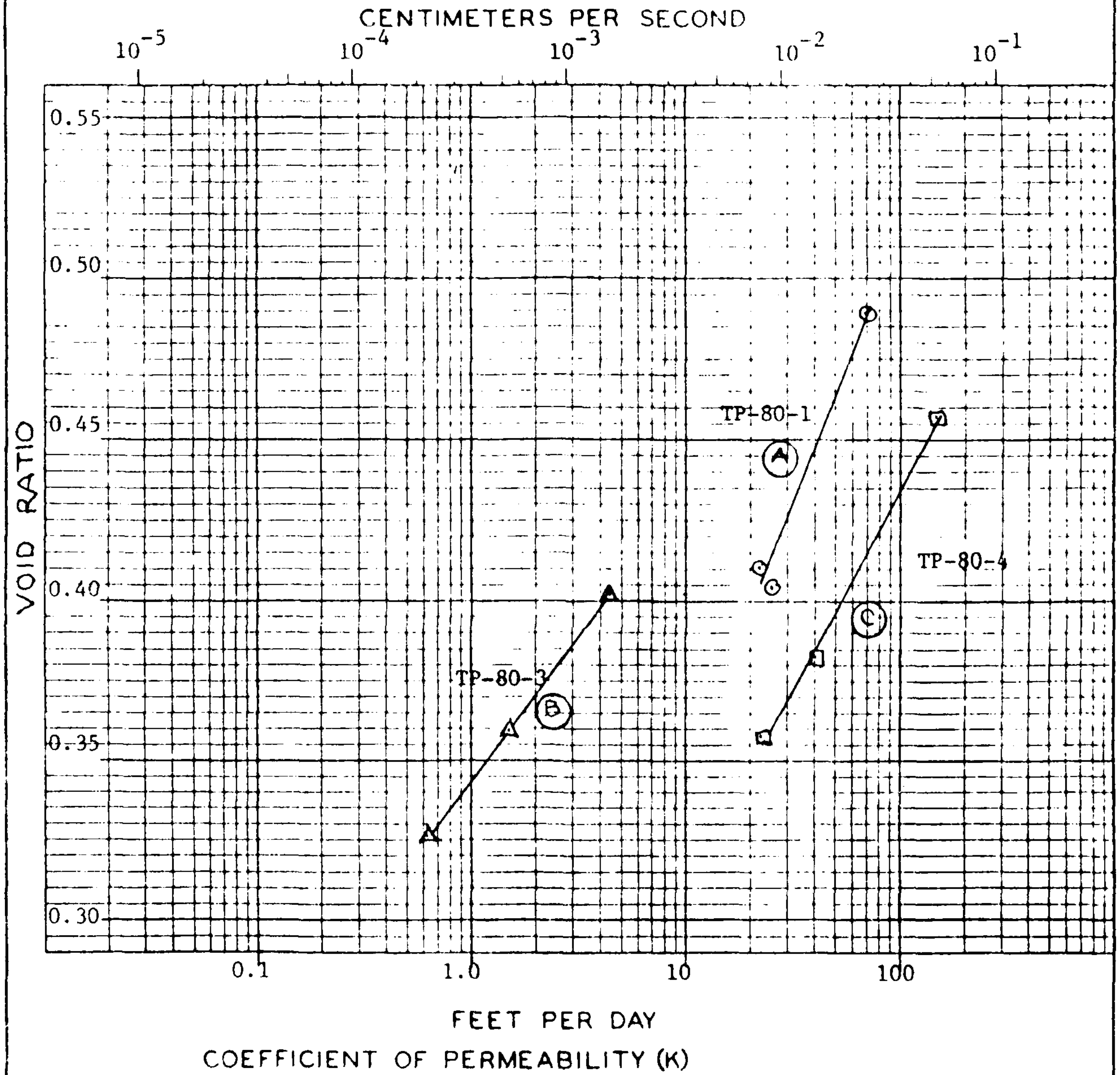
SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: Curve B tested in pressure permeameter

CURVE	SPECIMEN DIAM. IN	HT. IN.	MAX PARTICLE	CONDITION
A	6	6	3/4-in.	Remolded air-dry
B	6	12	3/4-in.	
C	6	6	3/4-in.	

DISTRICT: Los Angeles

PROJECT: RANCHO MIRAGE DAM

CURVE	DIV. NO	HOLE NO	F.S. NO.	DEPTH FROM	TO
A	74121	TP-80-1			
B	74123	TP-80-3			
C	74124	TP-80-4			

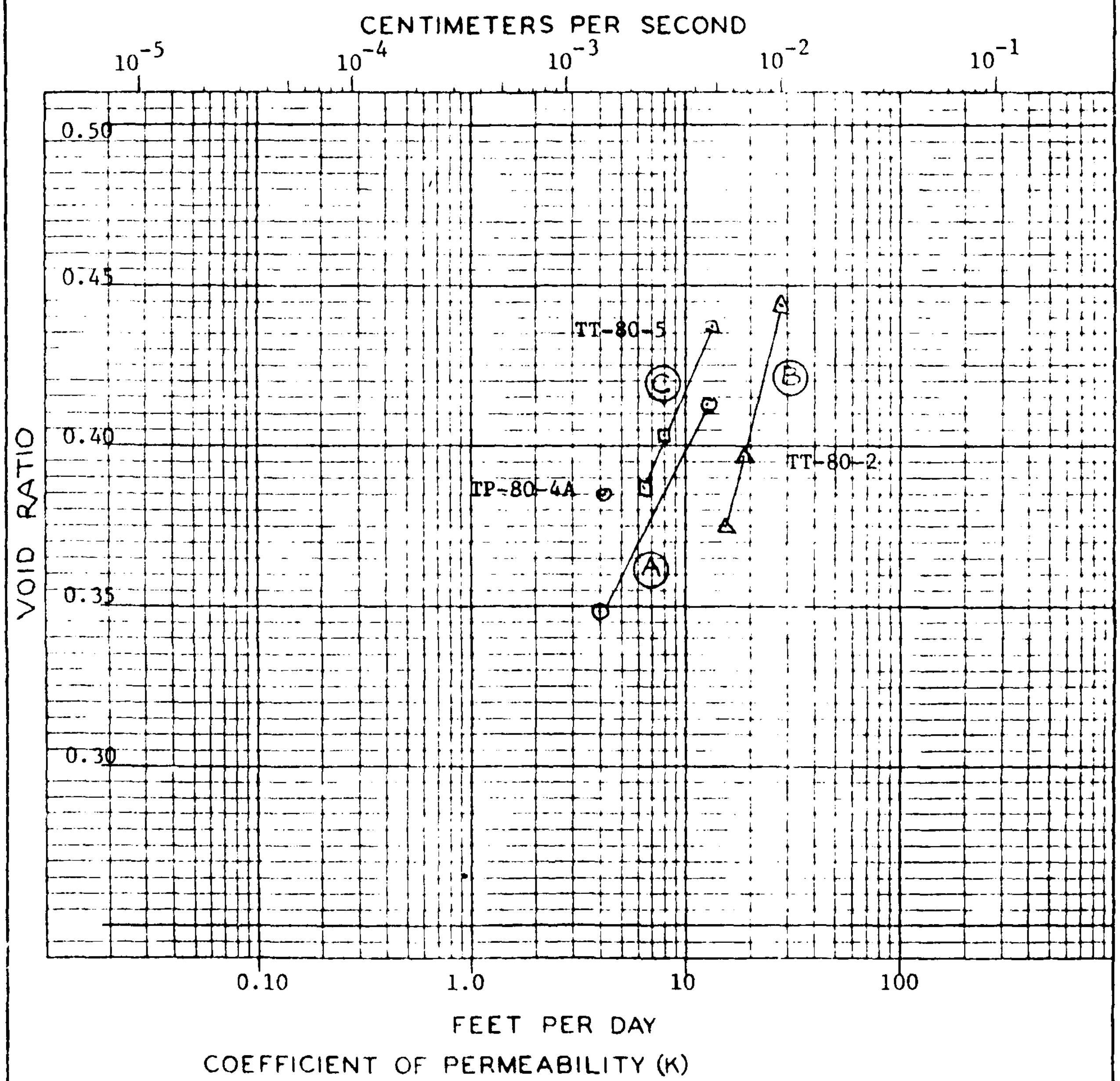
TESTED	COMPUTED	DRAWN	CHECKED
MT	MT		

SPD FORM 67 (CIVIL) 8 OCT 55 REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

CORPS OF ENGINEERS | U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: Cave B tested in pressure permeameter

CURVE	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
A	6.0	6.0	3/4-in.	Remolded air-dry
B	6.0	12	3/4-in.	
C	6.0	6.0	3/4-in.	

DISTRICT: Los Angeles

PROJECT: RANCHO MIRAGE DAM

CURVE	DIV. NO	HOLE NO	F.S. NO.	DEPTH FROM	DEPTH TO
A	74125	TT-80-4A			
B	74128	TT-80-2			
C	74126	TT-80-5			

TESTED	COMPUTED	DRAWN	CHECKED
MT	MT		

APPENDIX D

WHITEWATER RIVER BASIN, CALIFORNIA

MAGNESIA SPRING CANYON

DETAILED PROJECT REPORT FOR FLOOD CONTROL

RIVERSIDE COUNTY

APPENDIX D

ENGINEERING DESIGN

AND

COST ESTIMATES

U.S ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

SEPTEMBER 1983

APPENDIX D

ENGINEERING DESIGN AND COST ESTIMATES

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STRUCTURAL DESIGN

General. This section presents the feature design for the structural elements of the proposed flood control plan. The structural elements for this project include rectangular reinforced concrete spillway and channel walls, and reinforced concrete intake tower and outlet pipe at the debris basin.

References. All structures would be designed in accordance with applicable provisions of the following Engineering Manuals for Civil Works construction.

Reference	Date	Title
EM 1110-1-2101	November 1, 1963	Working Stresses for Structural Design
EM 1110-2-2103	May 21, 1971	Details of Reinforcement-Hydraulic Structures
EM 1110-2-2502	May 29, 1961	Retaining Walls
EM 1110-2-2902	March 3, 1969	Conduits, Culverts, and Pipes

Unit Design Stresses. Pertinent information on Unit Design Stresses used in the design of the proposed improvements is given in the following table.

TABLE 1

UNIT DESIGN STRESSES

Concrete:		
Ultimate compressive strength		
Cast-in-place structures other than culverts	f'c	= 3,000 psi
Culverts	f's	= 4,000 psi
Allowable compressive strength		
Flexure for retaining walls	fc	= 0.35 f'c = 1,050 psi
Flexure for culverts	fc	= 0.45 f'c = 1,800 psi
Shear		= 60 psi for f'c = 3,000 psi
		= 70 psi for f'c = 4,000 psi
Ratio n $= \frac{ES}{EN}$	n	= 9.3 for f'c = 3,000 psi
	n	= 8.0 for f'c = 4,000 psi
Modulus of elasticity	Ec	= 3,122,000 psi for f'c = 3,000 psi
	Ec	= 3,605,000 psi for f'c = 4,000 psi
Reinforcing Steel, Grade 40:		
Allowable tensile strength	fs	= 20,000 psi
Modulus of elasticity	ES	= 29,000 ksi

The weights and properties of soils are given in the geotechnical portion of this report.

Debris Basin. Structural design criteria for various elements of the basin, is described in the following paragraphs.

a. Intake Tower. The wall thickness would be determined from a stress analysis by applying the differential head of water between the inside and outside of the tower. In the determination of stability, the design load and

buoyancy of the structure as well as seismic forces would be considered. The tower would be supported by a spread footing which would be designed so that the resultant of the vertical and horizontal loads would fall within the middle third of the footing. When the seismic forces are considered, the resultant would be designed to fall within the middle half. The tower would be checked for two loading conditions: Condition I, when the reservoir is empty with seismic loading (seismic zone 4) and Condition II, when the reservoir is full to spillway crest elevation with no seismic loading. The possibility of an earthquake occurring simultaneous with Condition II is remote; therefore such a condition will be disregarded.

b. Spillway Walls.

(1) Upstream of the embankment. Walls upstream from the axis of the embankment would be designed according to the amount of backfill behind the wall. The first section at the entrance to the spillway would be designed for 5 loading conditions: Condition I, saturated backfill and an empty channel with a 1/3 increase in allowable stresses due to rapid drawdown; Condition II, drained backfill with an empty channel and normal stresses; Condition III, drained backfill plus construction equipment surcharge load with an empty channel. The allowable concrete and steel stresses of 25 percent above normal would be used for this condition; Condition IV, channel is full with passive pressure due to backfill counteracting the hydrostatic force in the channel. Condition V, loading assumes a free standing wall with a seismic force of 0.2g applies in either direction. An increase in allowable stresses of 33 percent would be included for this condition.

The design of the second section would be the same as above, except that Condition V would be omitted. Minimum channel face vertical reinforcing steel would be determined by one of the criteria as follows: (a) No. 4 bars spaced at 2 feet on centers, (b) 10 percent of the vertical steel in the earth face of the wall, (c) steel as required by Condition IV or V, whichever is greater.

(2) Downstream of embankment. The walls downstream from the axis of embankment are assumed to be outside of the zone of saturation; therefore, only drained earth backfill would be considered. The loading conditions for chute walls are given in paragraph b, section (1) above; however, only Conditions II, III, and IV would be used. A subdrainage system with perforated pipes would be provided at the spillway crest.

(3) Divider wall.

c. Reinforced Concrete Outlet Pipe. The 36-inch RCP under the embankment would be designed for Condition I (i.e., when the debris basin is empty) and Condition II (i.e., when the debris basin is full). Condition I loading would be as follows: (a) the vertical pressure equals 1.5 times the height of the fill times unit weight of the embankment and (b) the horizontal pressure equals 0.5 times the height of the fill times the unit weight of the embankment. The pipe would be designed for earthfill plus highway loading equivalent to HS 20-44 design loading to protect against damage from construction equipment. For Condition II loading, the water pressure over the conduit on the upstream side of the embankment would be considered. Reinforced concrete pipe would be encased in concrete. The design loads would be determined in accordance with EM 1110-2-2902 and a Safety Factor of 2.0 would be used. All pipe joints under the embankment of the debris basin shall be steel bell and spigot with gasket.

Rectangular Channel. The walls of the open rectangular reinforced concrete channel would be designed as L-Type or U-Type retaining walls. For L-Type retaining walls, the concrete invert slab between the wall footings would be 10-inch thick with a center mat of reinforcement consisting of 5/8-inch diameter steel bars at 12 inches on centers in each direction. The walls would be designed in pairs opposite each other with the wall footing abutting the 10-inch-thick invert slab. This type of design would prevent sliding. For U-Type retaining walls, the 10-inch-thick concrete invert slab would not be needed.

Both L-walls and U-walls would be designed for two loading conditions: Condition I (i.e., when the channel is empty), and Condition II (i.e. when the channel is full). For Condition I loading, earth pressure on the back of the wall would be determined in accordance with criteria contained Civil Works Engineer letter 64-7, 22 April 1964. Subject: "Construction Stresses in Retaining Walls". The lateral earth pressure due to a condition of drained backfill would be computed.

The triangle distribution of the horizontal earth pressure would be assumed in the design of the wall stem. Besides the earth pressure, a maximum loading of 200 psf due to construction equipment would be applied at the top of wall; the loading would be decreased by unit lateral earth pressure Kw at each foot of depth. The allowable stresses for concrete and steel under this loading condition would be increased by 25 percent. Friction with a coefficient equal to the tangent of 3/4 Ø (internal friction angle of the backfill material) would be assumed to act on the back of the walls. Straight-line distribution of soil pressure would be assumed in the design

wall footing. For Condition II loading, the hydrostatic pressure of 62.5 pounds per cubic foot on the channel side of the wall would be balanced by the passive lateral earth pressure acting on the back of the wall. Minimum reinforcing steel in the channel face of the wall would be the same as in paragraph b section (1) Spillway Walls.

AD-A150 305 WEST MAGNESIA CANYON CHANNEL CITY OF RANCHO MIRAGE RIVERSIDE COUNTY CALIF..(U) ARMY ENGINEER DISTRICT LOS ANGELES CA DEC 83 3/3

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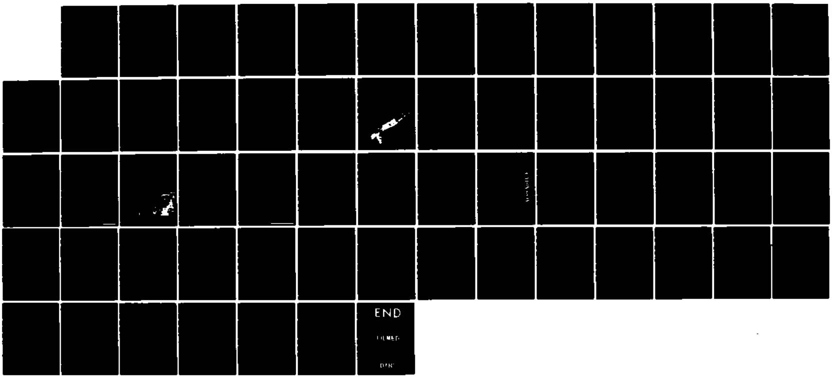
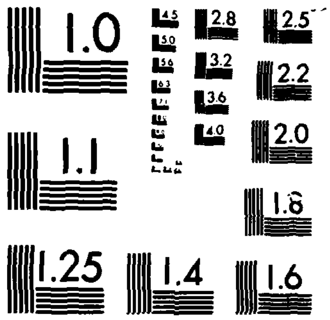




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

RECOMMENDED PLAN

General

West Magnesia Spring Canyon is located about 7 miles southeasterly of Palm Springs, California within the City of Rancho Mirage. The upstream canyon is undeveloped and is covered with sparse brush. The alluvial materials forming the floor and sides of the canyon erode readily and produce significant quantities of mineral debris. Flows from a major single storm event would result in debris-laden floods discharging from the mouth of the canyon. Deposition of debris would cause flows to cut new channels across the alluvial cone, constituting a flood and debris threat to the highly developed areas downstream from the mouth of the canyon.

The recommended improvements would consist of a debris basin and 7000 feet of reinforced-concrete channel from the debris basin to the Whitewater River. The debris basin would be designed to retain 150,000 cubic yards of debris; the spillway would be designed for the maximum probable flood peak discharge of 44,000 cfs to protect the embankment against overtopping. The channel design discharges, ranging from 6600 to 6800 cfs would conform to standard project flood peak discharges because--in the areas where this project-unit is located--such design is essential to preclude the dangerous effects of incomplete confinement of large floods. Unconfined floods could result in loss of life and major property damage in the intensively developed urban area concerned; and in damage to the flood-control channel itself. Because of these hazards, all similar projects located in Los Angeles County drainage area project have been so designed. Details of the major features of these improvements are given in the following paragraphs.

Debris Basin

General. The debris basin would be formed by excavation of the streambed and by the construction of an embankment from the excavated material. A rectangular concrete broad-crested spillway and a pool drain would be provided.

Embankment. The embankment would have a maximum height of 37 feet, a length of 800 feet, a top width of 20 feet, and slopes of 1 vertical on 3 horizontal. The top of the embankment would drain toward the spillway; a 4 inch thick layer of aggregate base course and 2 inch thick asphaltic concrete would be provided on the top of the embankment for access and drainage control. Paved gutters 6 feet wide would be provided at the junction of the basin embankment and the abutment slopes to prevent formation of gullies. A 4-inch thick concrete slab would be provided on the upstream slope of the embankment to minimize seepage. A downstream pervious drain blanket would be provided to control underseepage and through seepage. The downstream embankment face would be planted with shallow-rooted native vegetation (such as Atriplex canenis) where overbuild for the access road permits. Rock would placed along the toe of the downstream face of the embankment.

Spillway. The spillway would be a rectangular reinforced-concrete structure consisting of an approach channel, a control section and an outlet channel. The spillway would be 335 feet long and would be 190 feet wide at the control section; wall heights would range from 10 to 19 feet. The spillway would join the channel approximately 335 feet downstream from the top of the embankment. A subdrainage system would be provided under the spillway to relieve hydraulic uplift pressures.

Pool drain. The pool drain would consist of an ungated intake tower, an outlet conduit, and a diversion structure and conduit. The intake tower would be about 120 feet upstream from the axis of the embankment; the outlet conduit would extend downstream from the intake tower and would discharge into the spillway outlet channel through the diversion structure. The diversion structure and conduits would convey flows to the spillway and to the east side of the alluvial cone.

(a) Intake tower. The intake tower would be a circular reinforced-concrete structure. The top would be at least 1 foot above the assumed maximum debris elevation. The tower would be 5 feet in diameter (inside dimension) with 4 inch by 18 inch openings regularly spaced horizontally and vertically. A 3 foot by 3 foot grated manhole would be provided at the base of the tower for draining the basin and for cleaning process.

(b) Outlet Conduit. The outlet conduit would be concrete-encased reinforced-concrete pipe with a length of about 240 feet, a slope of about 5 percent, and an inside diameter of 36 inches.

(c) Diversion. The diversion structure and two diversion conduits would be reinforced concrete. The conduits from the diversion structure would both have diameters of 36 inches and a total combined length of about 700 feet. A flow restrictor plate will limit the total maximum discharge from the diversion conduits to 50 cfs. The conduits would terminate at the upstream end of the 20-acre mitigation area located along the east side of the cone between the mountain slopes and a levee to be constructed by the Coachella

Valley Water District. The diversion structure would also feature a 3-foot by 3-foot steel slide gate that would allow the closure of the pipes during maintenance.

Access Road. Access to the basin would be along the channel access road. A turn-around and small parking area would be provided at the base of the embankment. This road would be extended over the right abutment of the embankment to the basin bottom. The road would be graded to a minimum width of 16 feet and paved for a width of 12 feet to the top of the embankment. Paving would consist of a 4-inch thick layer of aggregate base course and 2-inch thick layer of asphaltic concrete. From the top of the embankment to the bottom of the basin, the road would be a minimum of 12 feet wide.

Fencing. A 4-foot chain-link and barbed-wire safety fence would be provided along the tops of the walls of the spillway. A 6-foot chain-link and barbed-wire security fence would be provided along the downstream toe of the embankment and would extend above each abutment to elevation 525. The fencing along the downstream embankment toe would include a locking double drive gate to restrict vehicle access to the debris basin. It would also include a 4' x 6' walk gate for pedestrian traffic traveling to the State ecological reserve. The pedestrian entrance gate would be located adjacent to the gate for vehicle traffic and would have set-in-place posts to preclude motorcycle access at all times. The gate would allow pedestrian access to be limited during the summer months when the California Department of Fish and Game closes the ecological reserve.

Channel

A reinforced-concrete channel 7000 feet long would be built between the downstream end of the spillway and the energy dissipator at the Whitewater River. Side drainage would enter the channel by concrete spillways at the tops of the concrete channel walls. The easterly berm along the channel would be paved with a 4-inch thick layer of aggregate base course.

Energy Dissipator. An energy dissipator would be constructed at the Whitewater River. The dissipator would be 516 feet of reinforced-concrete channel with baffle blocks.

Bank Protection. The banks of the Whitewater River at the outlet from Magnesia Spring Creek would be protected from erosion with stone revetment consisting of a 33 inch layer of rip-rap and a 15-inch layer of filler material.

Fencing. Both sides of the channel will be fenced for its entire length with chain link fencing. The westerly side of the channel right-of-way would be fenced with Helvie fencing which utilizes posts spaced 10 feet apart and three strands of smooth wire spaced in the following manner: bottom wire set 20 inches above the ground; middle wire set 15 inches above the bottom wire; and the top wire set 4 inches above the middle wire. The Helvie-type fence is required in consideration of the bighorn sheep in the area. A double-drive gate and 4' x 6' walk gate similar to those described in the Debris Basin section would also be provided in the channel fencing. The pedestrian walk gate would be located at a point along Mirage Road upstream of the intersection with Gorgonio Road. These gates would also control access as previously described. The specified location of the walk gate would ensure

that street parking and the entrance gate are contiguous and that the walking distance to the State ecological reserve would be minimized.

Guzzler/Adit. Enhanced water source(s) for big game would be provided in lower Magnesia Spring Canyon immediately upstream of Magnesia Falls as a first item of construction. As shown on plate D-8, the guzzler consists of a buried concrete box with a water-collection apron and a valve and drinking trough. An adit consists of a large down-sloping water storage tunnel (dimensions 6' x 6' x 21') with a water-collecting surface as shown on plate D-8. Fifteen thousand dollars ($15,000) is alloted for construction of a guzzler or an adit.

Mitigation Area.

A 20-acre mitigation area would be located along the east edge of the alluvial cone between the toe of the mountains and a levee to be built by the Coachella Valley Water District. The levee is planned to be constructed prior to construction of the Corps project. Development of the mitigation area would include planting of native vegetation (seedlings), watering and maintenance for up to 2 years to aid establishment of seedlings, and no-trespassing signs designating the area as protected wildlife habitat that would be posted every 200 feet along the top of the 3,500-foot-long levee. The numbers and types of seedlings to be planted and the specifics of the planting, watering, and other maintenance efforts will be addressed by an environmental contract in 1984.

REAL ESTATE REQUIREMENTS

General. Construction of the debris basin and channel would require about 29 acres of land of which 16 acres would be allocated for the debris basin and access road, and the other 13 acres would be utilized for the channel. Approximately 20 acres along the east side of the cone downstream of the debris basin would be acquired for mitigation. Project land costs were determined by using a conservative value of $5000 per acre for 70 acres. No appraisals have been obtained as project justification is not sensitive to land cost and conservative values have been assumed.

Acquisition. In accordance with the authorizing act, the Coachella Valley Water District will acquire all rights-of-way, including temporary construction easements, for the construction of the project. All acquisition will be completed prior to the initiation of construction.

BRIDGES AND UTILITY RELOCATIONS

There are no bridge or utility relocations required for the project. The State Route 111 bridge at station 16+57.5 can accommodate the proposed channel with no modification. All known utilities in the vicinity of the project are located on the bridge.

DIVERSION AND CONTROL OF WATER DURING CONSTRUCTION

General. Based on climatological data the dry seasons occur between the months of September to November and March to July. The major construction would generally be limited to these periods, making diversion and control of water requirements minimal. The debris basin construction would be planned for the fall to minimize effects on the bighorn sheep.

COST ESTIMATES

General. Total estimated cost for the project as recommended in this report is $8,279,000, of which $4,000,000 is a Federal cost and $4,279,000 is a non-Federal cost. The detailed estimated costs for the project based on March 1983 price levels is shown on table 3. Unit prices are based on costs prevailing in March 1983 for work of this nature in the Los Angeles area and in the vicinity of the site. A summary of the detailed estimated of first cost for the selected plan of improvement is given in table 2. The cost estimate for the proposed improvements includes construction, engineering and design, supervision and administration, right-of-way, and contingencies.

TABLE 2

Summary of Estimated Costs (March 1983 price levels)

Cost Acct. No.	Item	Amount
06	Fish and Wildlife Facilities	$ 185,000
09	Channel & Debris Basin	6,401,000
20	Reservoir Staff Gages	1,000
30	Engineering and Design	466,000
31	Supervision and Administration	388,000
	Subtotal, Construction Cost	$7,441,000
01	Lands and Damages	388,000
02	Relocations	0
	TOTAL PROJECT FIRST COSTS	$7,829,000
	Detailed Project Report	450,000
	TOTAL FLOOD CONTROL COSTS	$8,279,000
	Annual Operation and Maintenance Cost	$ 72,000

TABLE 3

DETAILED FIRST COST ESTIMATE FOR IMPROVEMENT
UNDER THE SELECTED PLAN
(March 1983 price levels)

Cost Acct. No.	Description	Quantity	Unit	Unit Cost	Amount
06	Fish and Wildlife Facilities:				
	Wildlife guzzlers	1	Job	LS	14,000
	Mitigation Area:				
	Native plants	1,700	Ea	11.00	19,000
	Planting & Establishment of plants	1	Job	LS	122,000
	Posts & Signs	1	Job	LS	6,000
	Contingencies (15%)				24,000
	Subtotal, Fish and wildlife Facilities				185,000
09	Channel & Debris Basin				
	Embankment:				
	Care and diversion of water	1	Job	LS	11,000
	Clearing and grubbing	9	AC	1,100.00	10,000
	Excavation, debris basin	96,000	CY	2.20	211,000
	Excavation, foundation	68,000	CY	2.20	150,000
	Excavation, abutment	1	Job	LS	80,000
	Compacted fill, random	140,000	CY	2.20	308,000
	Drain material	6,500	CY	11.00	72,000
	Aggregate base course	125	CY	16.00	2,000
	Grouted stone inlet	1	Job	LS	133,000
	Concrete facing slab	1	Job	LS	244,000
	Gutters	1	Job	LS	12,000
	Access road:				
	Compacted fill	6,800	CY	2.20	15,000
	Aggregate base course	175	CY	16.00	2,800
	A.C. paving	900	Ton	42.00	38,000
	6-foot chain link fence	900	LF	7.80	7,000
	Spillway:				
	Excavation	37,000	CY	2.20	81,000
	Compacted fill	26,000	CY	2.20	57,000
	Concrete, cutoff wall	45	CY	111.00	5,000
	Concrete, invert	2,500	CY	67.00	168,000
	Concrete, wall	700	CY	100.00	70,000
	Cement	15,000	CY	7.00	105,000
	Reinforcing steel	300,000	LB	0.44	132,000
	Fencing (4-foot)	860	LF	5.55	4,800
	Subdrainage	1	Job	LS	28,000

TABLE 3 (Continued)

Cost Acct. No.	Description	Quantity	Unit	Unit Cost	Amount
	Intake tower, Diversion-structure, and Drain pipe:				
	Intake tower	1	Job	LS	16,600
	Excavation	2,850	CY	2.20	6,300
	Compacted fill	900	CY	2.20	2,000
	36" R.C.P.	240	LF	50.00	12,000
	Concrete, Cradle	170	CY	188.00	32,000
	Cement	1,030	CWT	7.00	7,200
	Diversion structure	1	Job	LS	24,600
	Diversion outlet	1,000	LF	50.00	50,000
	Channel				
	Clear and grub	16	AC	1,100.00	17,600
	Diversion and control of water	1	Job	LS	11,000
	Earthwork				
	Excavation	110,000	CY	2.20	242,000
	Compacted fill, Channel	60,000	CY	2.20	132,000
	Concrete				
	Invert	2,600	CY	67.00	174,000
	Footings	5,000	CY	67.00	335,000
	Walls	4,670	CY	100.00	467,000
	Baffle Blocks	315	CY	166.00	52,000
	Cement	70,000	CWT	7.00	490,000
	Reinforcing steel	1,445,000	Lbs	0.44	636,000
	Chain link fence (6')	13,000	LF	7.00	90,000
	3' strand wire fence	6,500	LS	5.55	36,000
	Aggregate base course	1,000	CY	16.00	16,000
	Sidedrains	2	Ea	3,300.00	6,600
	Stone	17,000	Tons	20.00	340,000
	Grouting stonework	3,250	CY	44.00	143,000
	Excavation	40,000	CY	2.20	88,000
	Fill	21,000	CY	2.20	46,000
	Beautification	1		LS	197,000
	Contingencies (15%)				835,470
	Subtotal, channel & Debris Basin				6,401,000
20	Reservoir Staff gages	6	Ea	140.00	840
	Contingencies (15%)				160
	Subtotal, Reservoir staff gages				1,000
	Total, Channel, Debris Basin & Appurtenances				6,587,000

TABLE 3 (Continued)

Cost Acct. No.	Description	Quantity	Unit	Unit Cost	Amount
30	Engineering and Design				
	Plans and Specifications				$335,000
	Engineering during Construction				$111,000
	Subtotal, Engineering & Design				$466,000
31	Supervision and Administration				388,000
	TOTAL CONSTRUCTION COST				$7,441,000
01	Lands and Damages				388,000
02	Relocations				0
	TOTAL PROJECT FIRST COST				$7,829,000
30	Detailed Project Report				450,000
	Total Flood Control Costs				$8,279,000
	FEDERAL COST				$4,000,000
	NON-FEDERAL COST				$4,279,000

SCHEDULE FOR DESIGN AND CONSTRUCTION

The time required to prepare the plans and specifications for this project is approximately one year. This phase of the project could be initiated, pending approval of this report and receipt of funds. Construction would take approximately 9 months to complete. The construction of the debris basin would be scheduled in the fall and channel could be accomplished during the Spring or Fall months.

Excavation of the debris basin and construction of the embankment and upstream 1000 feet of the spillway and channel would be scheduled to the maximum extent practicable, to avoid the period from 15 June through 30 September in order to minimize adverse impacts of construction noise and activity on the bighorn sheep. Construction of the portion of the channel adjacent to the Rancho Mirage Elementary School would be scheduled, to the maximum extent practicable, to avoid school hours.

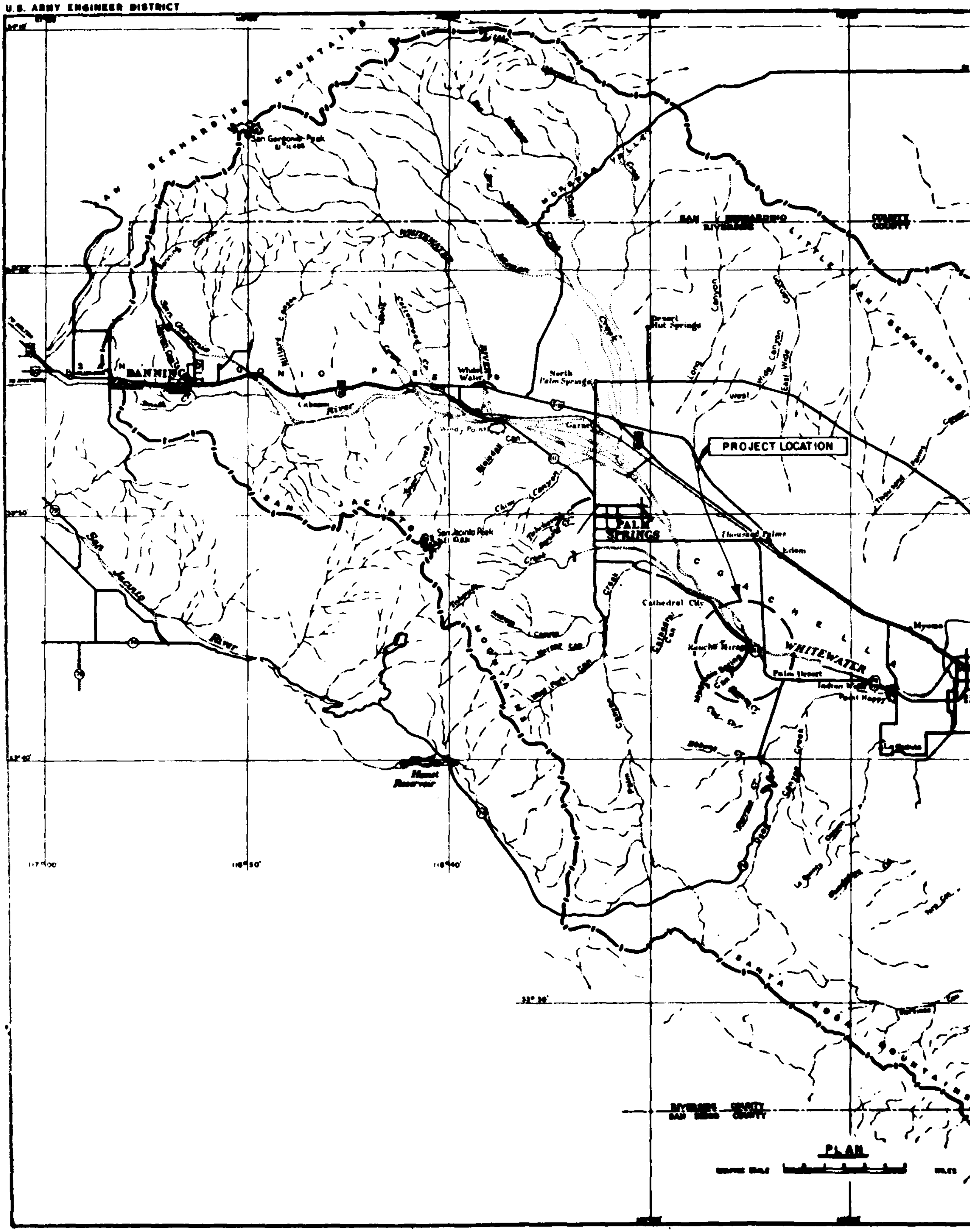
The guzzler/adit would be a first item of construction. Planting of the mitigation area would be scheduled to be accomplished during the period from October to April in order to insure a reasonable rate of planting success.

OPERATION AND MAINTENANCE

The Coachella Valley Water District will operate and maintain the completed facilities. Because maintenance and operation costs are to be projected for 100 years from the completion of the project, maintenance cost would include replacing 2 inches of concrete invert every 25 years as well as periodic removal of debris from debris basin in addition to routine maintenance. Debris removal costs of $11 per cubic yard are considered typical. The amount of debris accumulation is estimated to average 4000 cubic yards annually. Based on these figures, an average annual operation and maintenance charge of $72,000 was estimated for the project.

REPRODUCED AT GOVERNMENT EXPENSE

U.S. ARMY ENGINEER DISTRICT



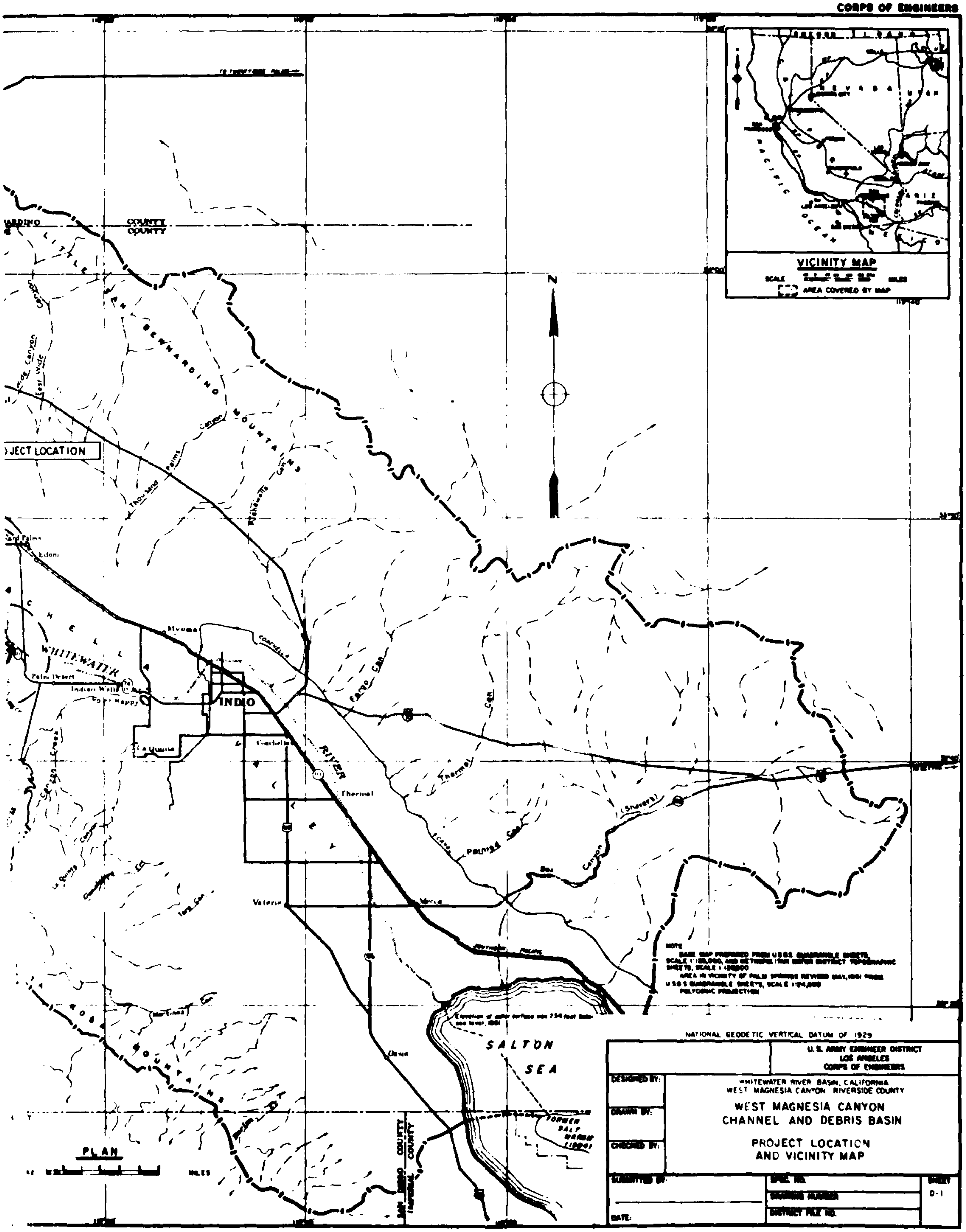
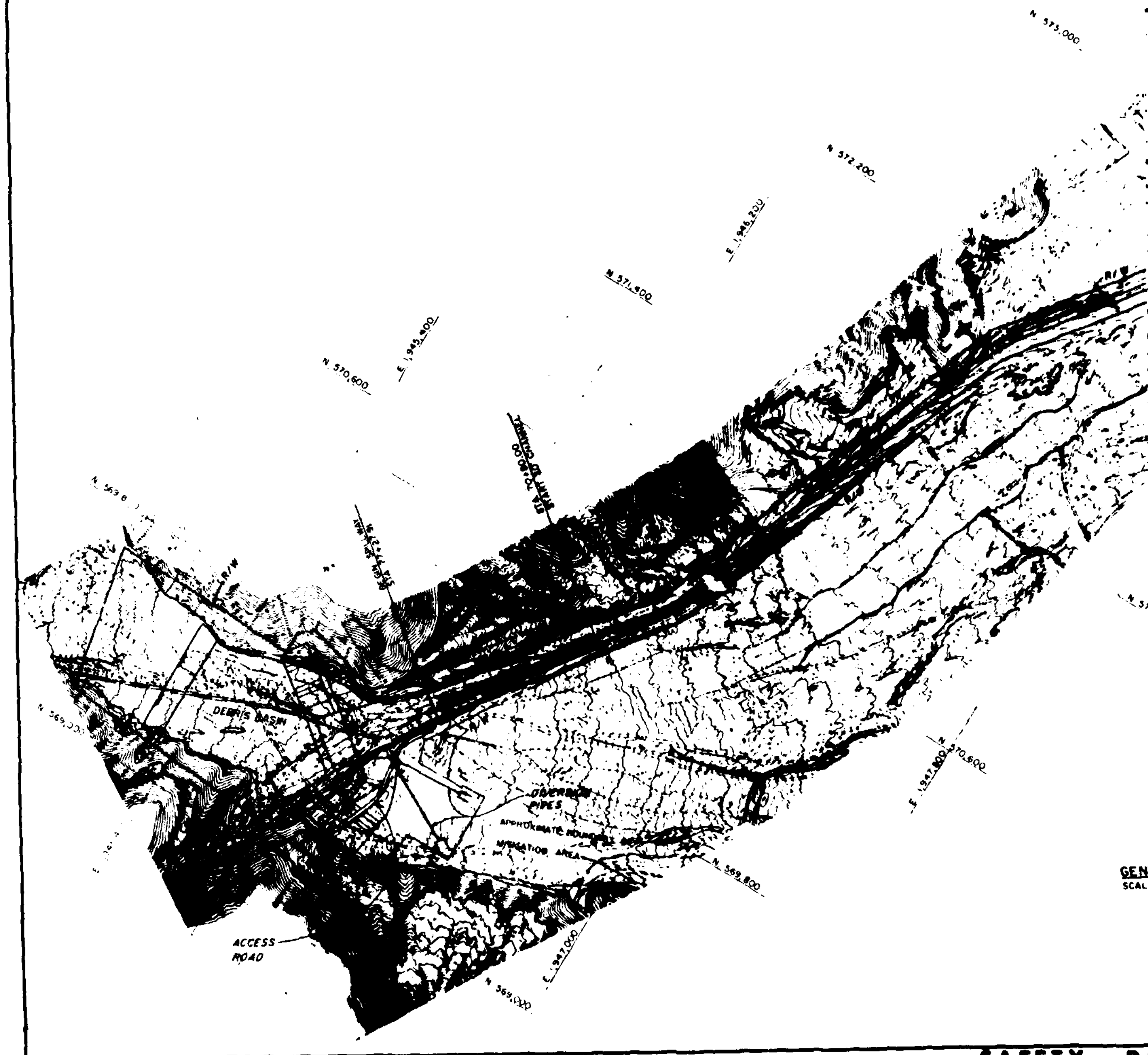

CORPS OF ENGINEERS
VICINITY MAP
AREA COVERED BY MAP
PACIFIC OCEAN
NEVADA
UTAH
COUNTY
COUNTY
LITTLE SAN BERNARDINO MOUNTAINS
PROJECT LOCATION
Thousand Palms Canyon
Pushawalla Canyon
East Wide
Wide Canyon
N
Edom
Myoma
Coachella
WHITEWATER
Palm Desert
Indian Wells
Point Happy
INDIO
La Quinta
Coachella
RIVER
Fargo Can
Thermal
Thermal Can
Painted Can
Box Canyon
(Shavers)
Mecca
Valerie
La Quinta
Guadalupe Ca
Toro Can
Martinez Can
Oasis
Southern Pacific
SALTON SEA
SAN DIEGO COUNTY
IMPERIAL COUNTY
MOUNTAINS
FORMER SALT MARSH (1860)
PLAN
MILES
NOTE
NATIONAL GEODETIC VERTICAL DATUM OF 1929
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
DESIGNED BY:
DRAWN BY:
CHECKED BY:
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON RIVERSIDE COUNTY
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN
PROJECT LOCATION
AND VICINITY MAP
SUBMITTED BY:
DATE:
SPEC. NO.
DRAWING NUMBER
DISTRICT FILE NO.
SHEET
D-1

PLATE D-1

II

VALUE ENGINEERI



SAFETY PA

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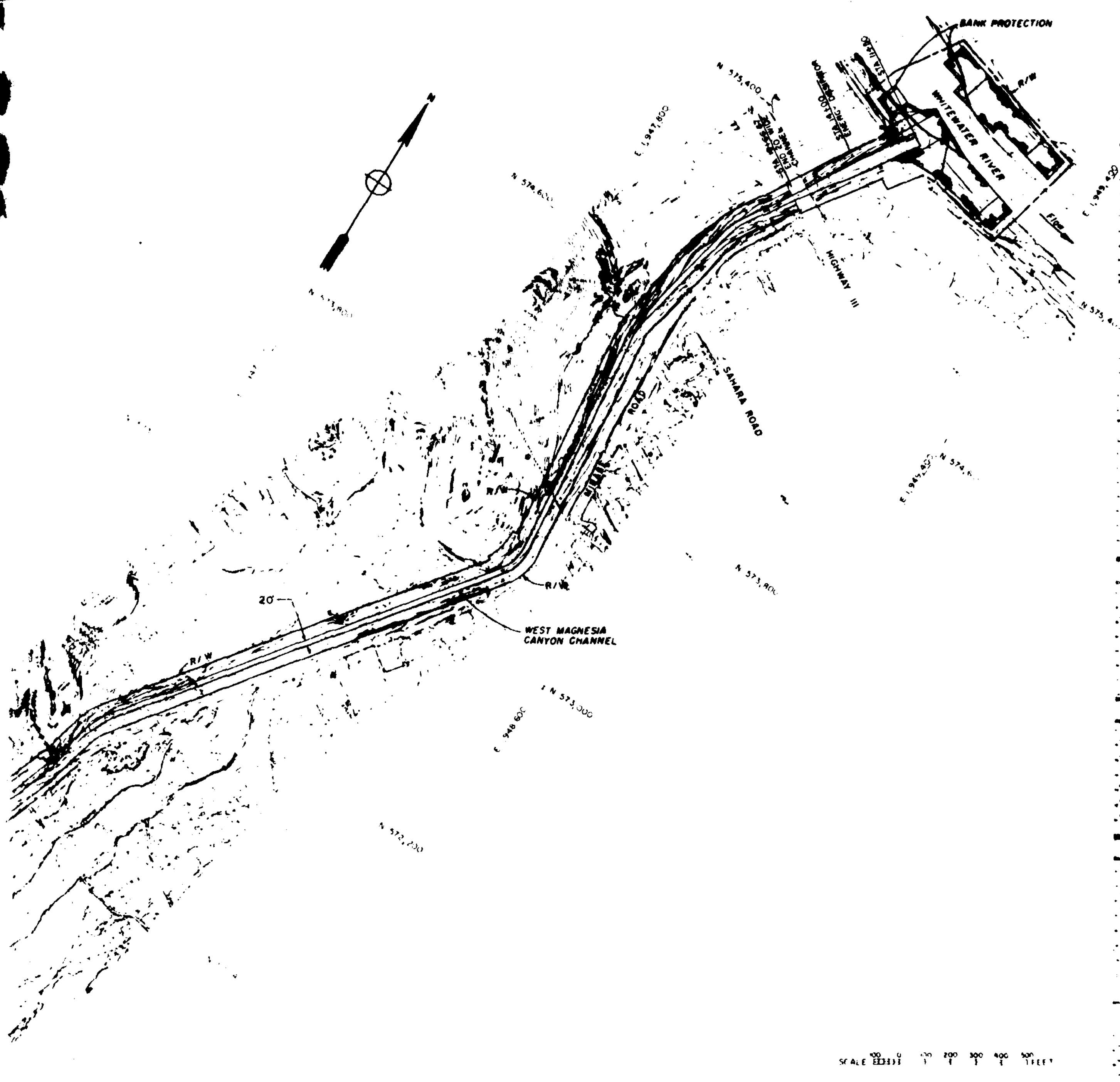


GENERAL PLAN
SCALE 1 IN = 200 FT

NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPRO

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

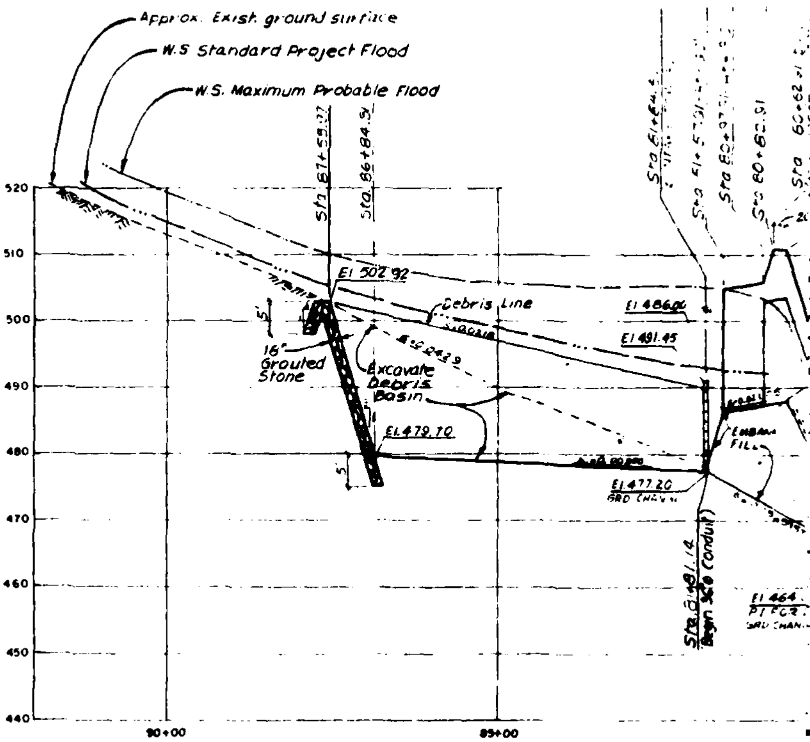
DESIGNED BY
DRAWN BY
PENUELA
CHECKED BY

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

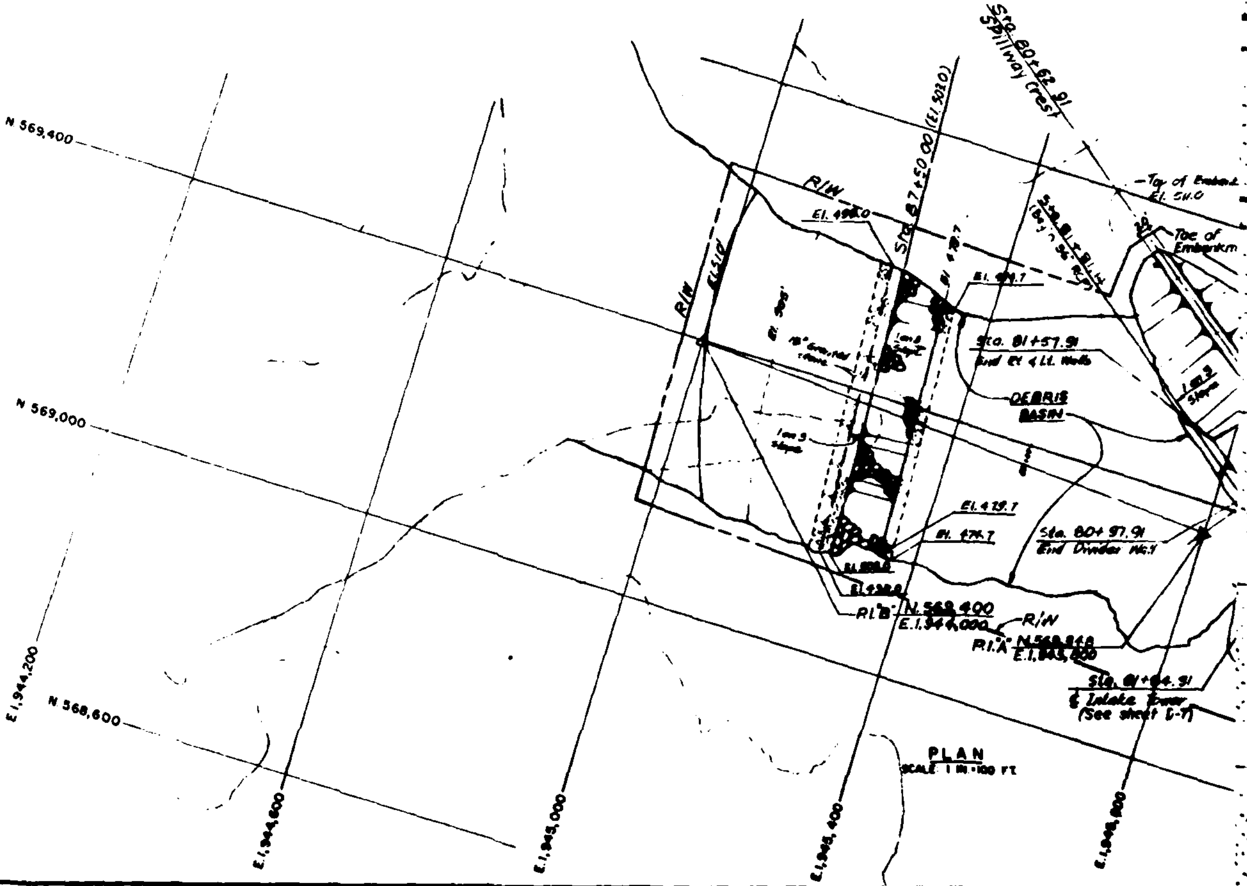
GENERAL PLAN

SUBMITTED BY:
DATE APPROVED
SPEC NO DACW09-
DISTRICT FILE NO



PROFILE
VERT SCALE 1 IN = 10 FT
HORIZ SCALE 1 IN = 100 FT

HYDRAULIC ELEMENTS														
STATION TO STATION		STATION	SLOPE		Q	Dc UPS-DS	n=0.014				n=0.012			
							Da RANGE		VEL. RANGE		Da RANGE		VEL. RANGE	
80+62.91	79+47.91	transition	0.208700		44,000	varies	11.91	6.43	19.55	47.25	11.91	6.43	19.55	47.25
79+47.91	77+27.91	"	0.049465		"	"	6.43	8.00	47.25	49.58	6.43	7.91	47.25	50.37
80+62.91	79+47.91	transition	0.208700		6800	3.35 3.66	3.35	1.47	10.37	35.63	3.35	1.48	10.39	38.18
79+62.91	77+27.91	"	0.049465		"	3.86 4.51	1.47	2.01	35.63	31.65	1.48	1.91	38.18	34.69
77+27.91	72+12.91	begin spillway	0.030108		"	4.51 12.28	2.01	9.14	31.65	25.79	1.91	7.99	34.69	31.30
72+12.91	70+80.00	transition	0.037417		"	12.29 15.0[illegible]	9.14	11.57	25.79	28.5[illegible]	7.99	9.99	31.30	33.02



PLAN
SCALE 1 IN = 100 FT

SAFET

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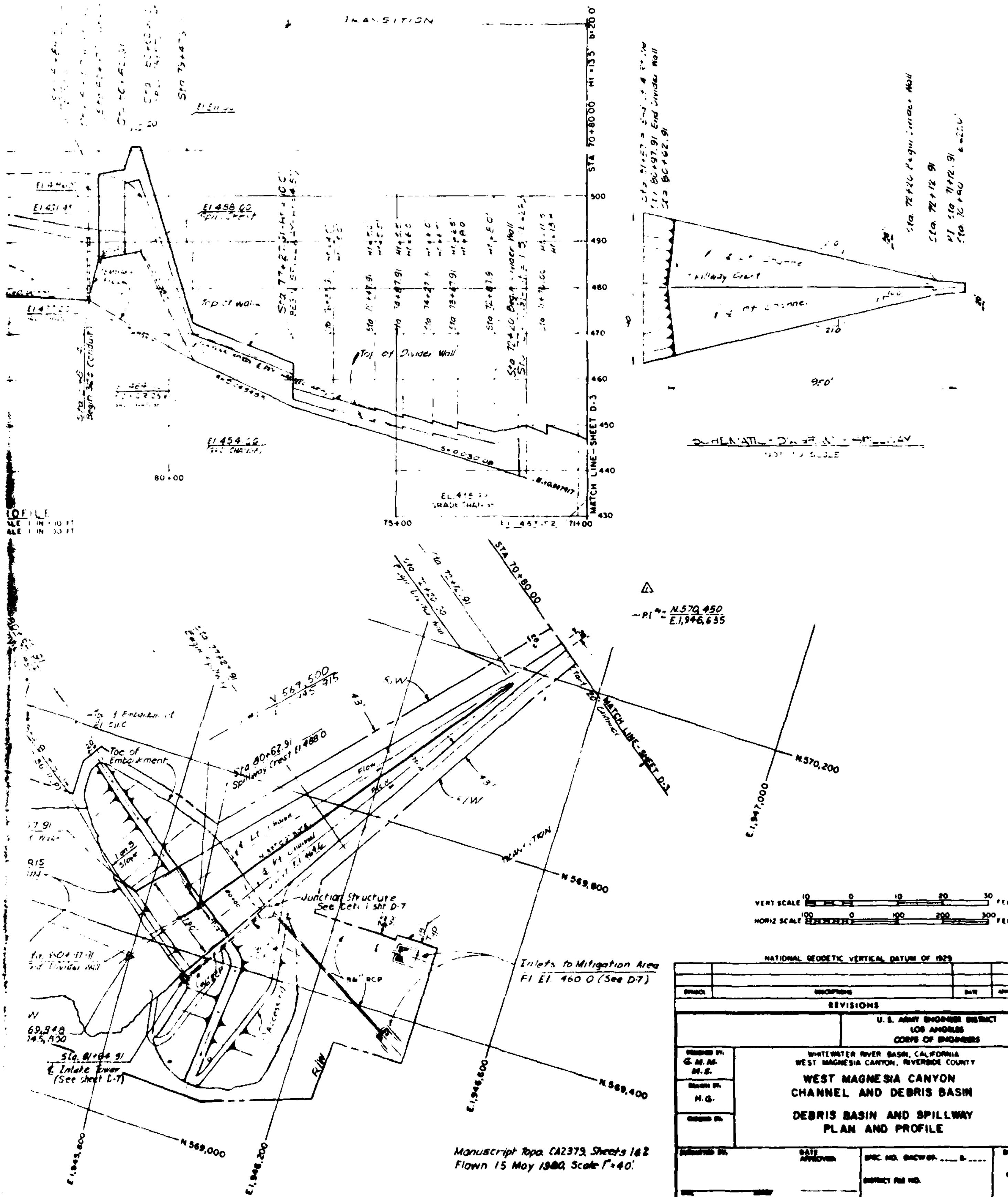
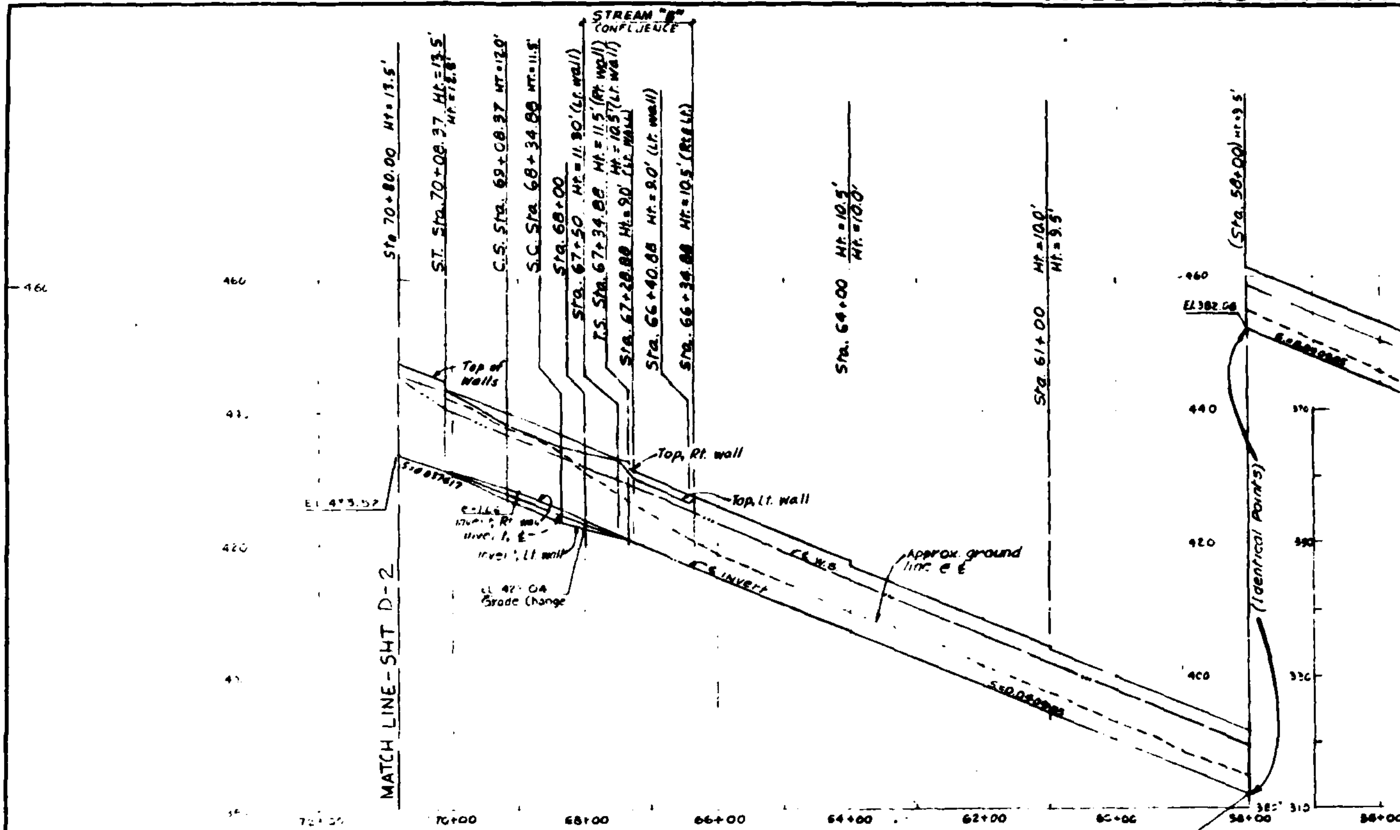
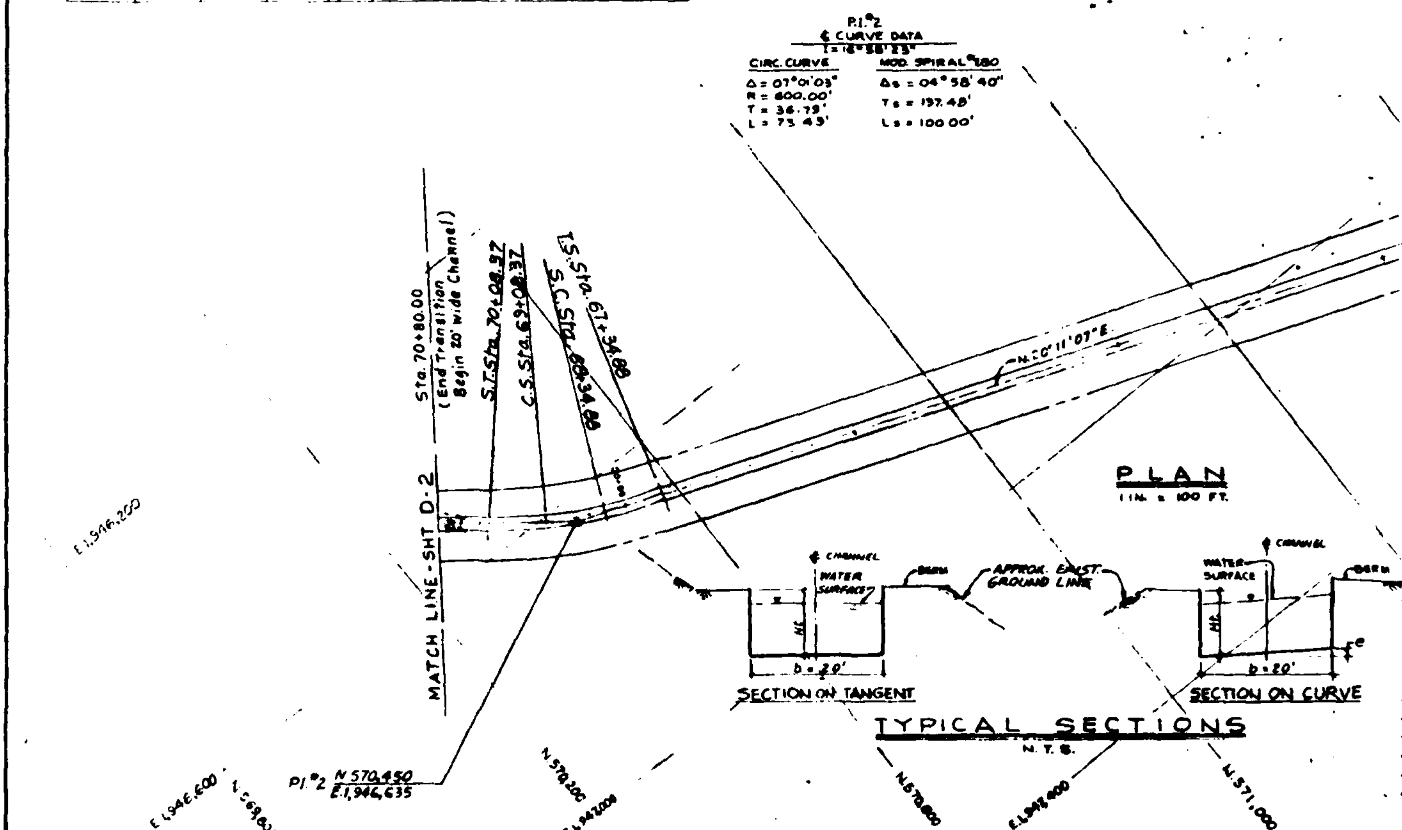

TRANSITION
STA 70+80.00
MATCH LINE - SHEET D-3
El 458.00 Spill Crest
Top of wall
Top of Divider Wall
El 454.00
80+00
75+00
71+00
PROFILE
SCHEMATIC DIAGRAM OF SPILLWAY
Spillway Crest
950'
Toe of Embankment
Sta 80+62.91 Spillway Crest El 488.0
Flow
R/W
N 570,200
N 569,800
N 569,400
N 569,000
E 1,947,000
E 1,946,600
E 1,946,200
E 1,945,800
-PI N 570,450 E 1,946,635
Junction Structure See Detail sht D-7
Inlets to Mitigation Area Fl El. 460.0 (See D-7)
Sta 81+64.91 ℄ Intake Tower (See sheet D-7)
36" RCP
Manuscript Topo. CA2379, Sheets 1&2
Flown 15 May 1980, Scale 1"=40'
VERT SCALE
HORIZ SCALE
NATIONAL GEODETIC VERTICAL DATUM OF 1929
REVISIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN
DEBRIS BASIN AND SPILLWAY
PLAN AND PROFILE
G.M.M.
M.S.
H.G.

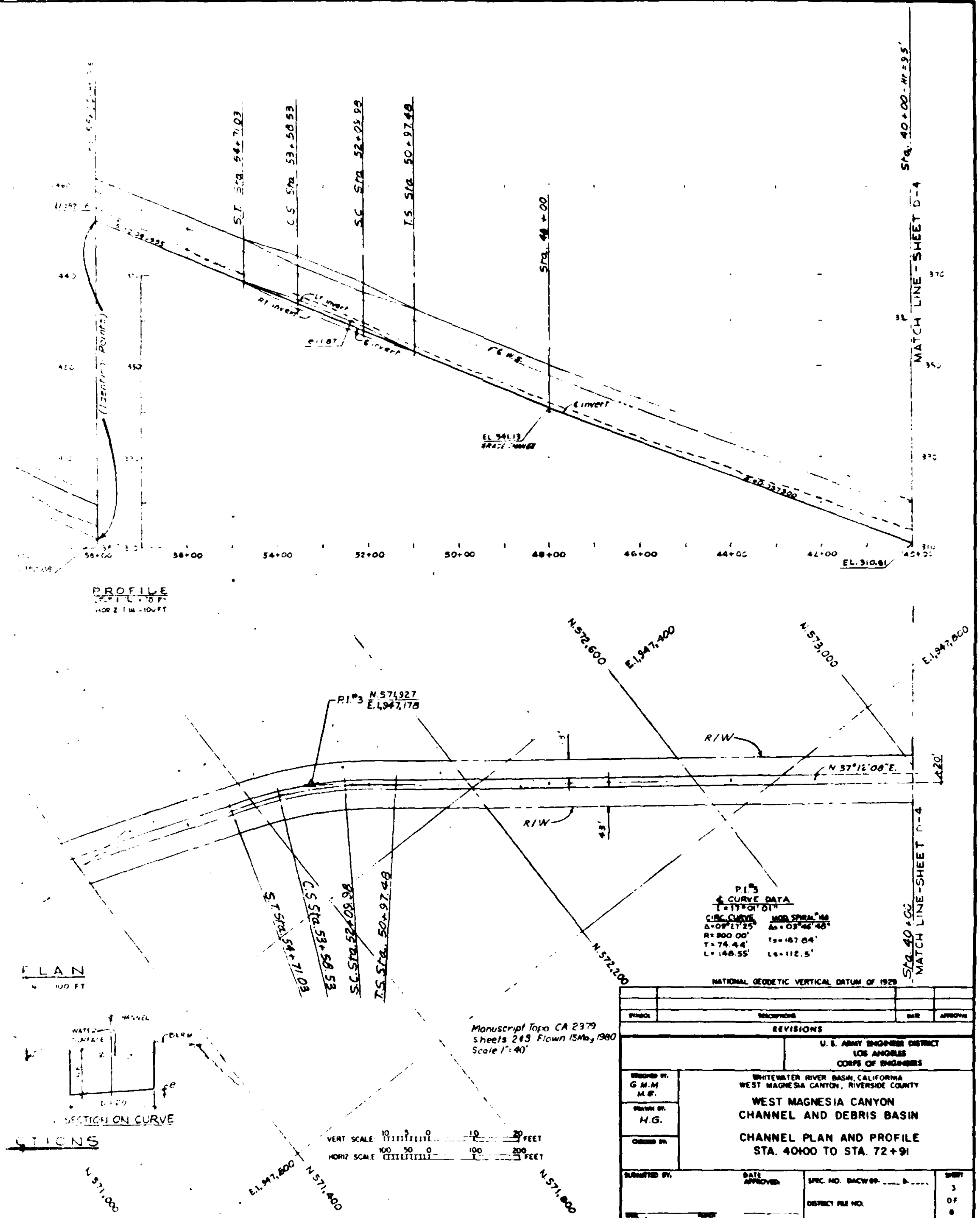
PLATE D

II



HYDRAULIC ELEMENTS														
STATION TO STATION		STATION	SLOPE	Q	Dc UPS-DS	n=0.014				n=0.012				
						Dn RANGE		VEL. RANGE		Dn RANGE		VEL. RANGE		
70+80.00	68+00.00	20 rectangular	0.037417	6600	15.02	11.57	9.03	28.54	28.10	9.09	8.53	32.02	40.88	
68+00.00	48+00.00	"	0.040955	"	"	9.03	7.19	38.10	52.07	8.53	6.77	40.98	57.22	
48+00.00	37+50.00	"	0.037900	"	"	7.19	7.23	52.07	51.64	6.77	6.77	57.23	57.23	





NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: G.M.M. M.E.
DRAWN BY: H.G.
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

CHANNEL PLAN AND PROFILE
STA. 40+00 TO STA. 72+91

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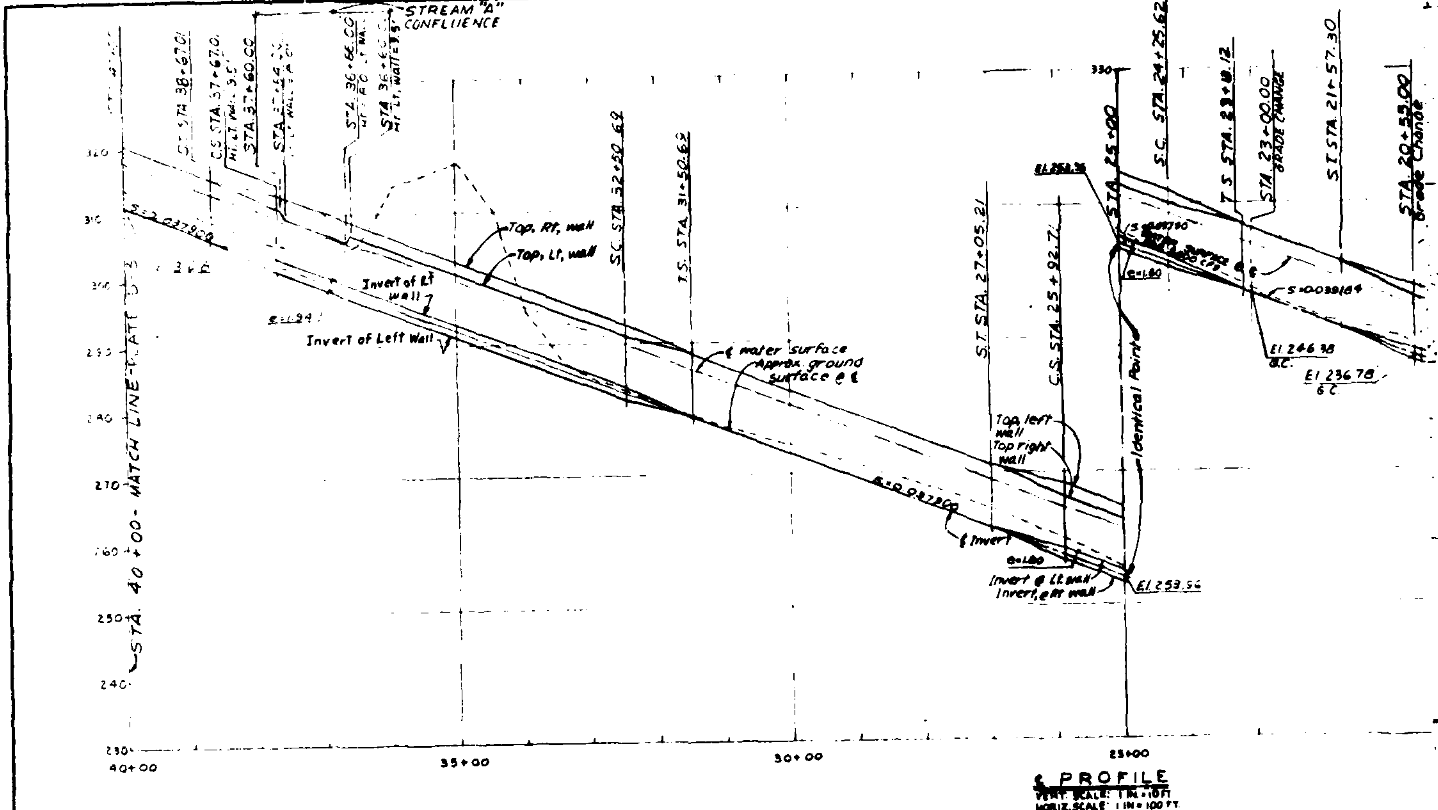
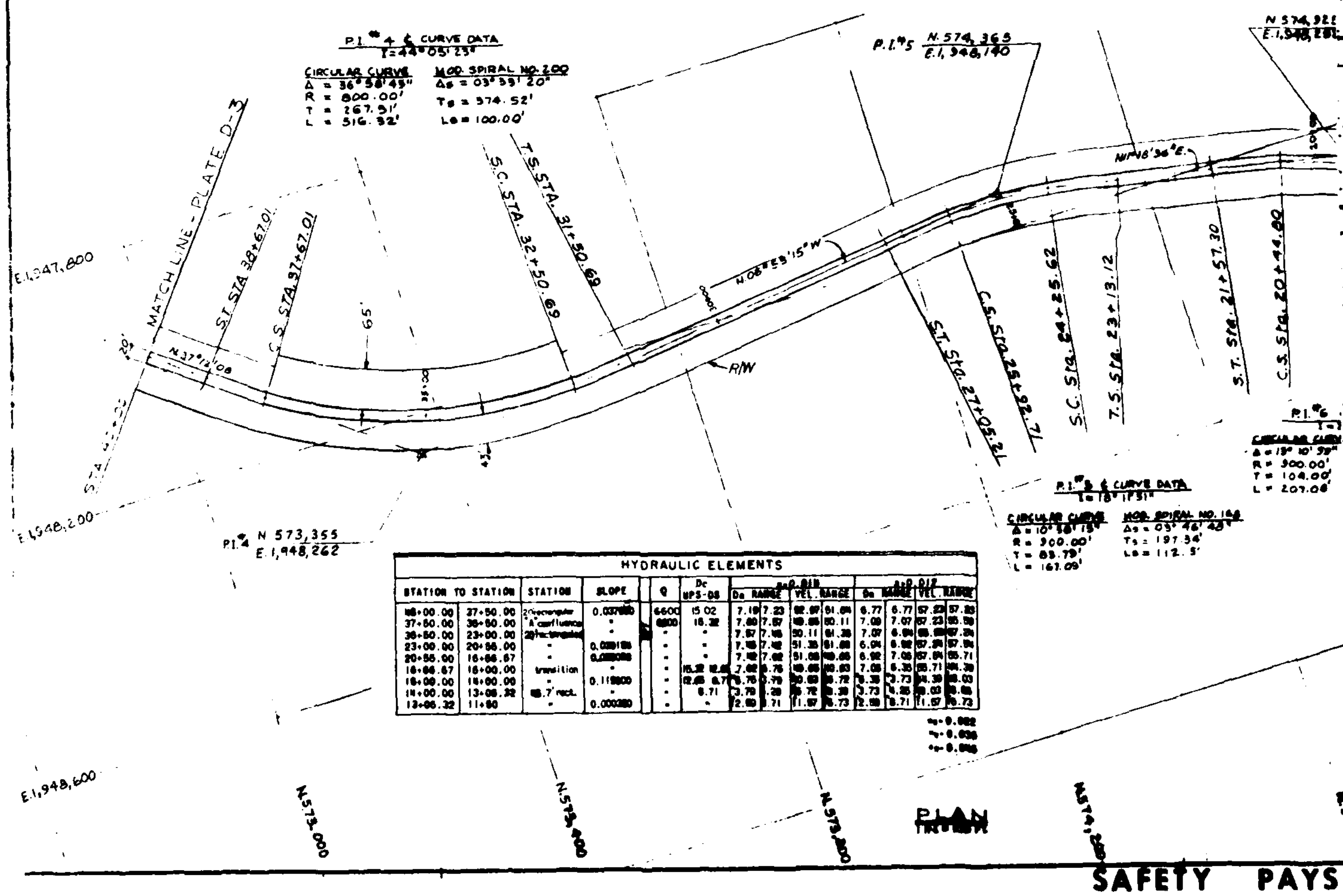
SHEET 3 OF 8

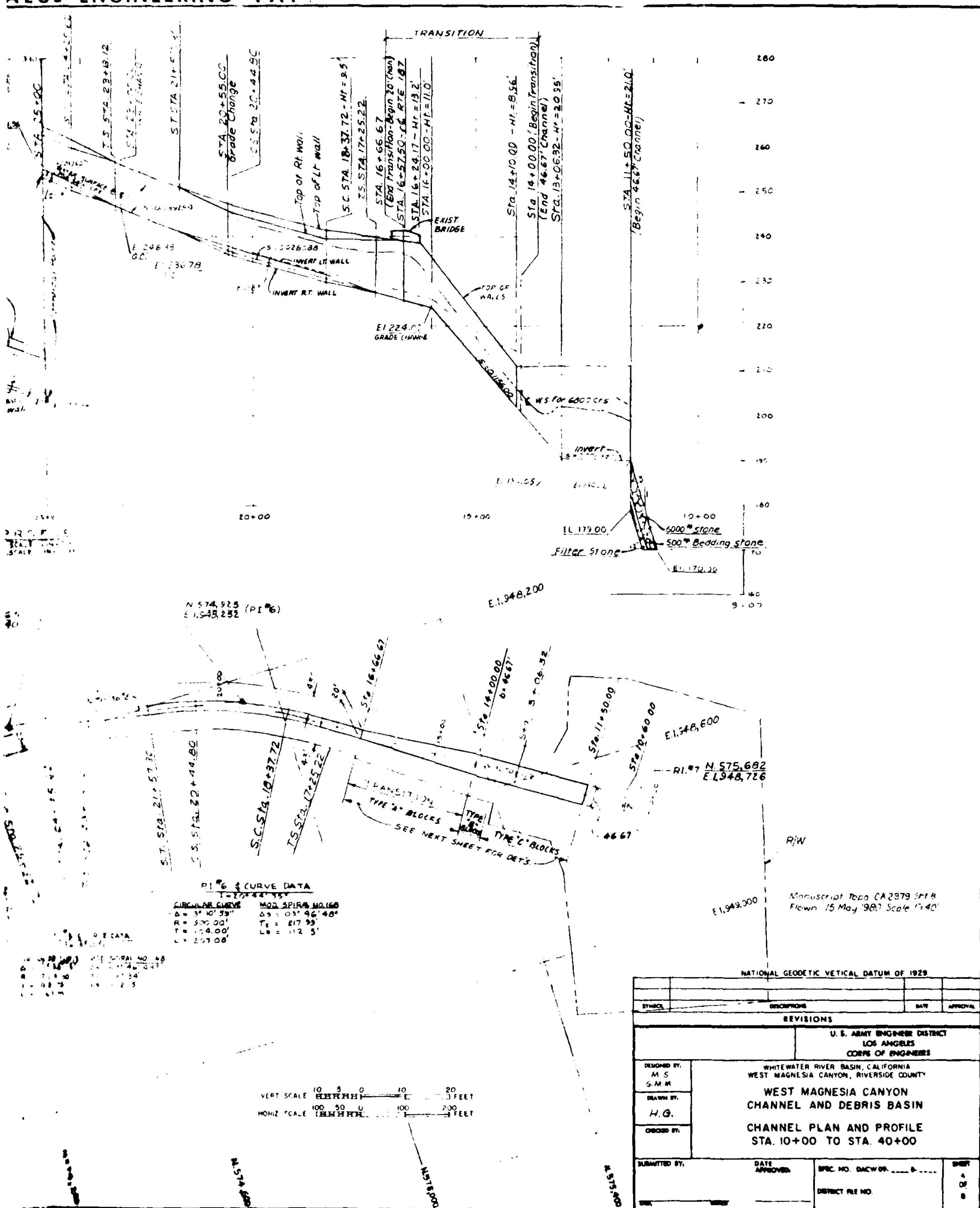
VALUE ENGINEERING





HYDRAULIC ELEMENTS														
STATION	TO STATION	STATION	SLOPE	Q	Dc UPS-08	n=0.018 Dn RANGE		VEL. RANGE		n=0.012 Dn RANGE		VEL. RANGE		
[illegible]+00.00	37+50.00	2@rectangular	0.037[illegible]	6600	15.02	7.19	7.23	[illegible]	51.[illegible]	6.77	6.77	57.2[illegible]	57.[illegible]	
37+50.00	36+50.00	"A" confluence	"	[illegible]	16.32	7.[illegible]	7.57	[illegible]	50.11	7.09	7.07	57.23	55.50	
36+50.00	23+00.00	2@rectangular	"	"	"	7.57	7.[illegible]	50.11	51.3[illegible]	7.07	6.94	[illegible]	57.[illegible]	
23+00.00	20+55.00	"	0.00[illegible]	"	"	7.[illegible]	7.42	51.35	51.[illegible]	6.94	6.92	57.[illegible]	57.94	
20+55.00	16+66.67	"	0.0[illegible]	"	"	7.42	7.02	51.08	[illegible]	6.92	7.06	57.94	55.71	
16+66.67	16+00.00	transition	"	"	15.32 12.[illegible]	7.02	[illegible].76	[illegible]	[illegible]	7.06	6.35	55.71	[illegible]	
16+00.00	14+00.00	"	0.11[illegible]	"	12.65 8.71	[illegible].76	[illegible]	[illegible]	[illegible].72	[illegible]	3.73	[illegible]	[illegible]	
14+00.00	13+06.32	[illegible].7' rect.	"	"	8.71	3.79	[illegible]	[illegible].72	[illegible]	3.73	[illegible]	[illegible]	[illegible]	
13+06.32	11+50	"	0.000[illegible]	"	"	2.[illegible]	[illegible].71	11.57	[illegible].73	2.[illegible]	8.71	11.57	[illegible].73	

SAFETY PAYS



NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: M.S. G.M.M.
DRAWN BY: H.G.
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

CHANNEL PLAN AND PROFILE
STA. 10+00 TO STA. 40+00

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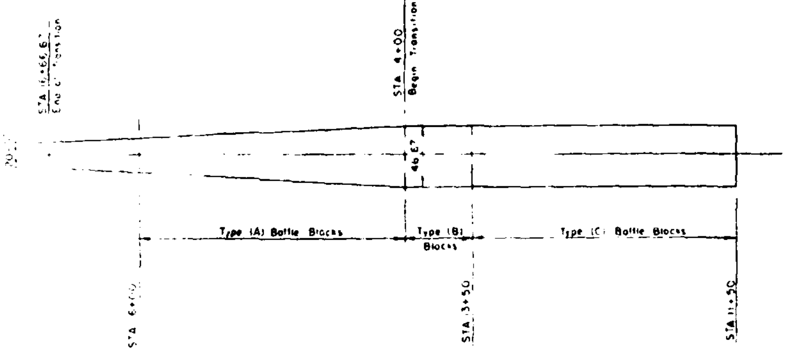
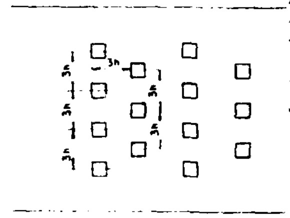
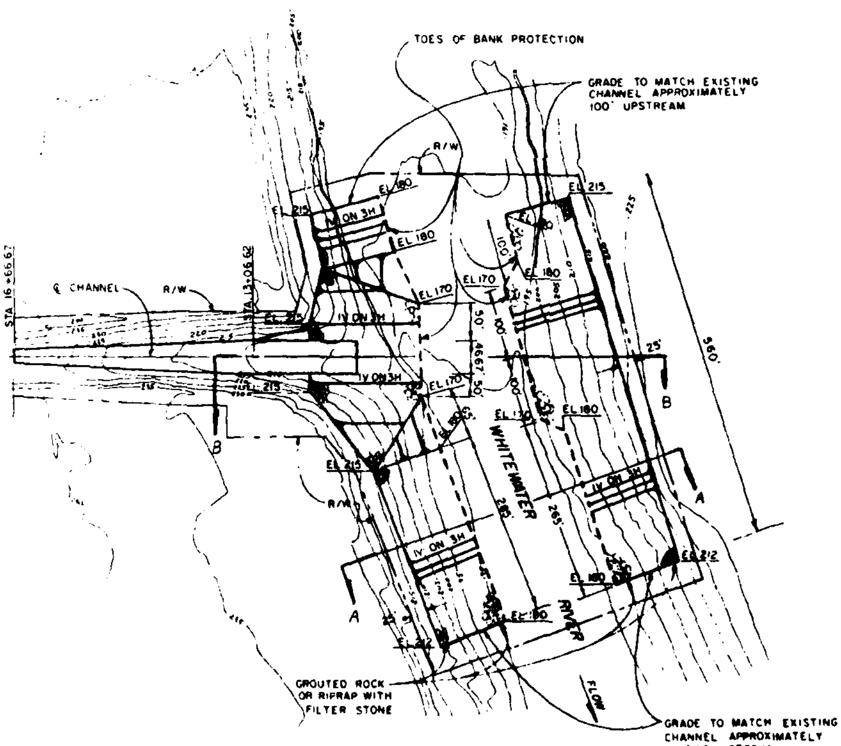
PLATE D-4



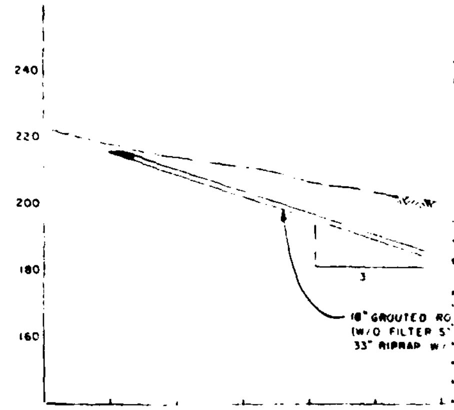
PLAN - BAFFLE BLOCK LOCATION
SCALE 1 IN = 50 FT

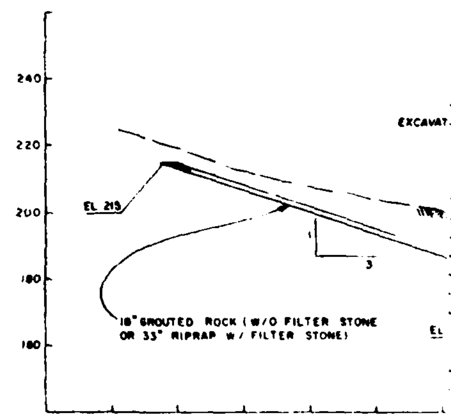


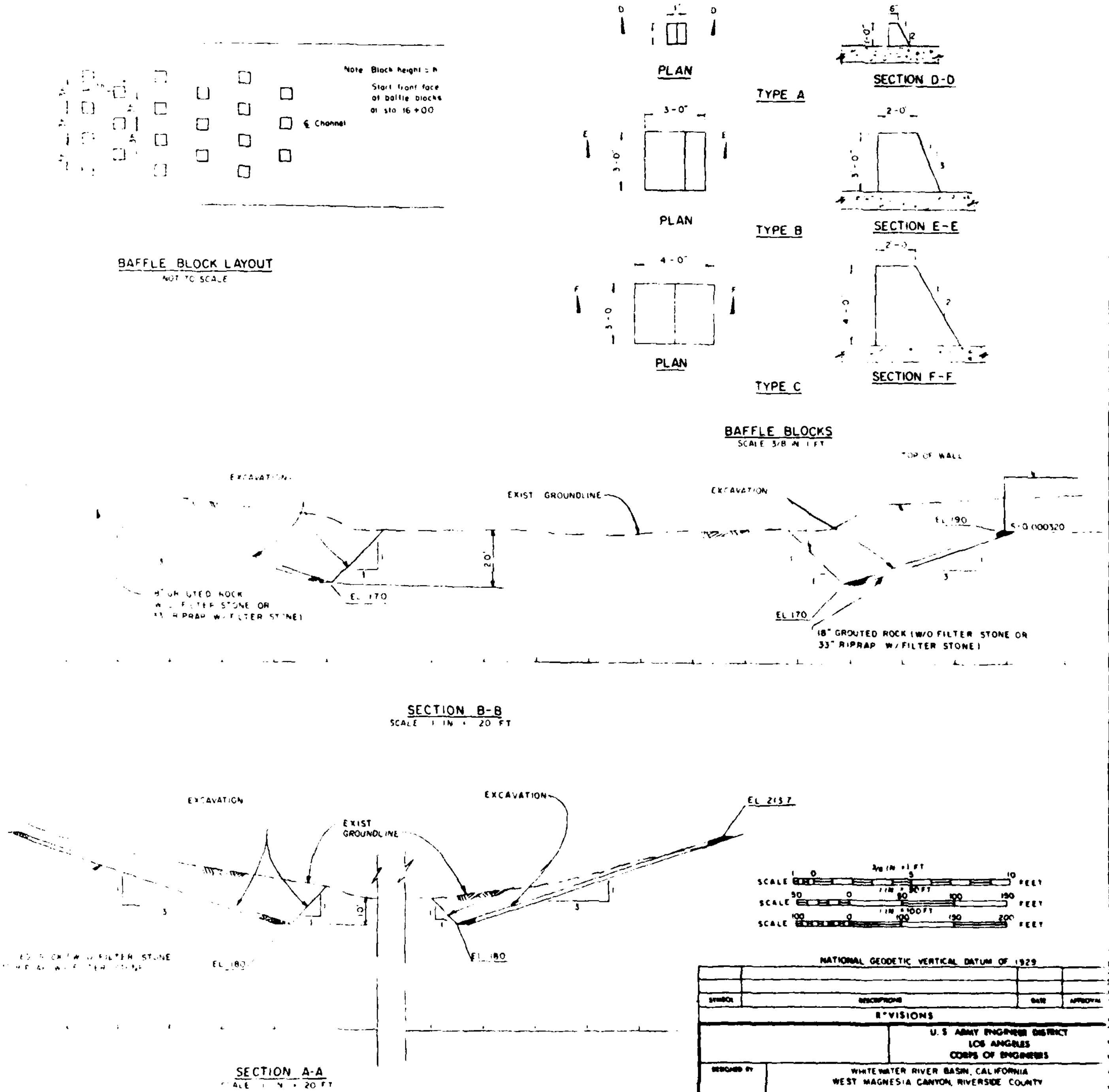
BAFFLE BLOCK L–
NOT TO SCALE



BANK PROTECTION
SCALE 1 IN = 100 FT







SCALE 3/8 IN = 1 FT, 0–10 FEET
SCALE 1 IN = 20 FT, 50–150 FEET
SCALE 1 IN = 100 FT, 100–200 FEET

NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

ENERGY DISSIPATOR
AND
BANK PROTECTION

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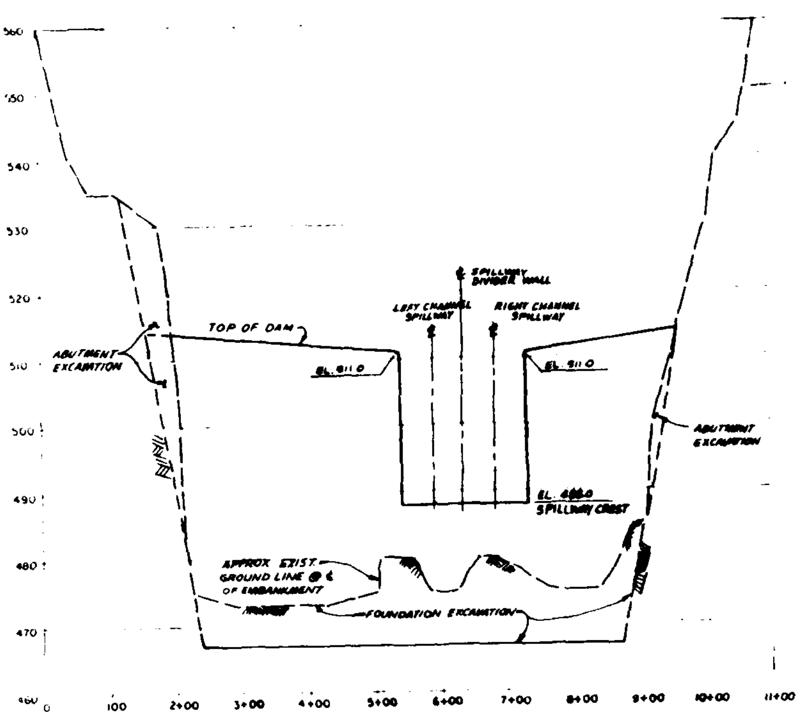
DISTRICT FILE NO.

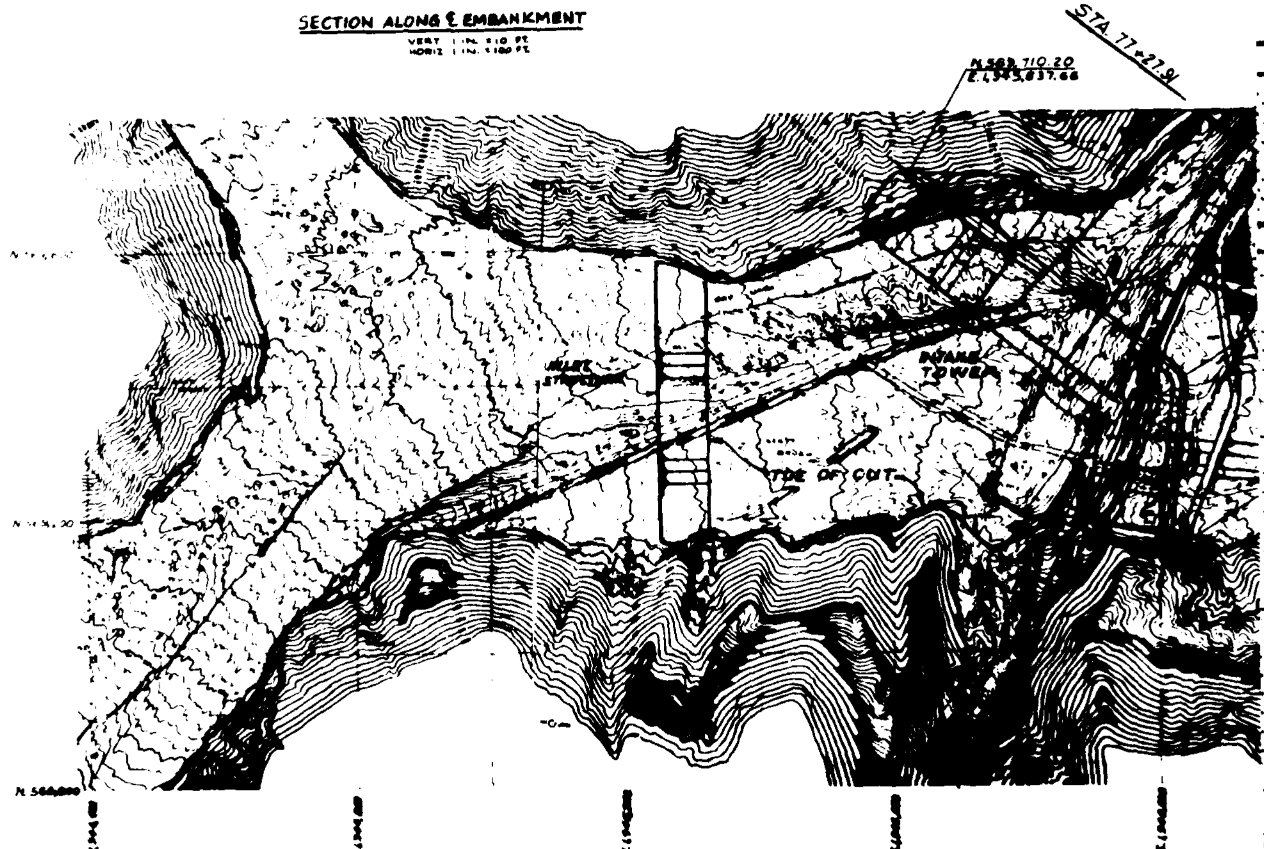
PLATE D

II

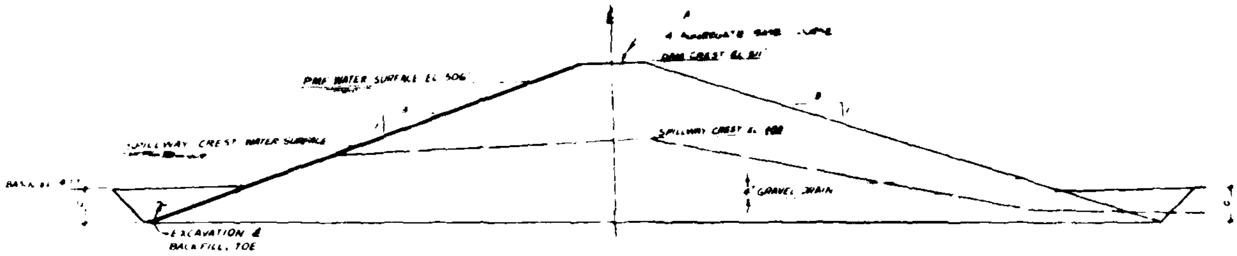


SECTION ALONG ℄ EMBANKMENT

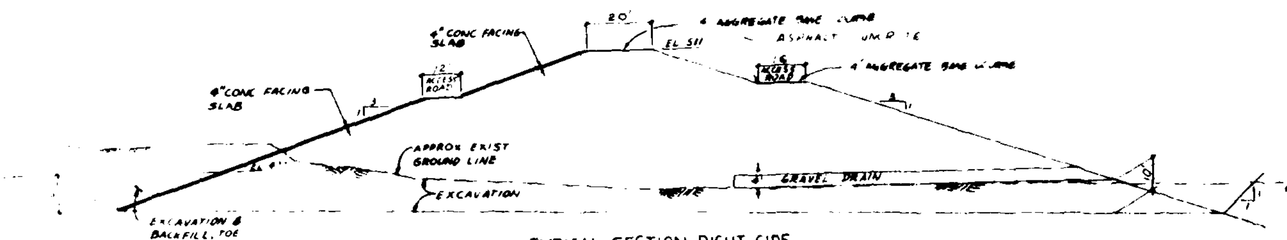
VERT. 1 IN. = 10 FT.
HORIZ. 1 IN. = 100 FT.



PLAN

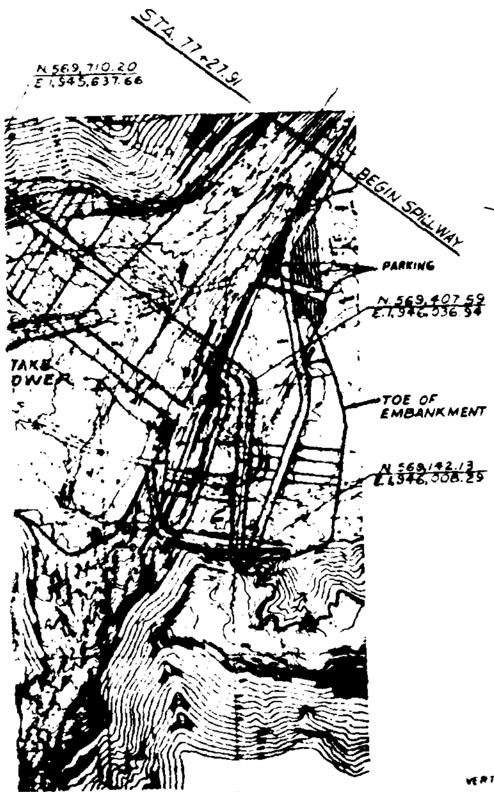


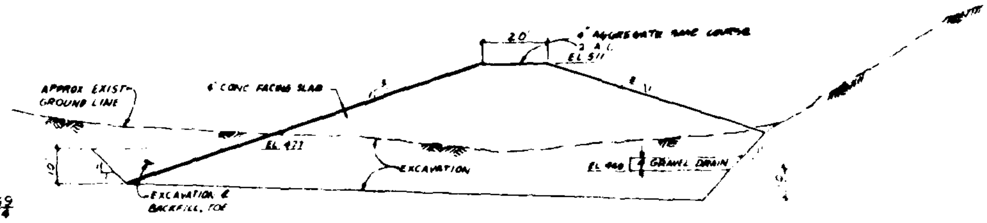
EMBANKMENT SECTION



TYPICAL SECTION-RIGHT SIDE

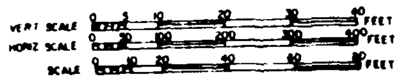
SCALE 1 IN = 20 FT





TYPICAL SECTION-LEFT SIDE

SCALE 1 IN = 20 FT



NATIONAL GEODETIC VERTICAL DATUM OF 1929

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
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WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY

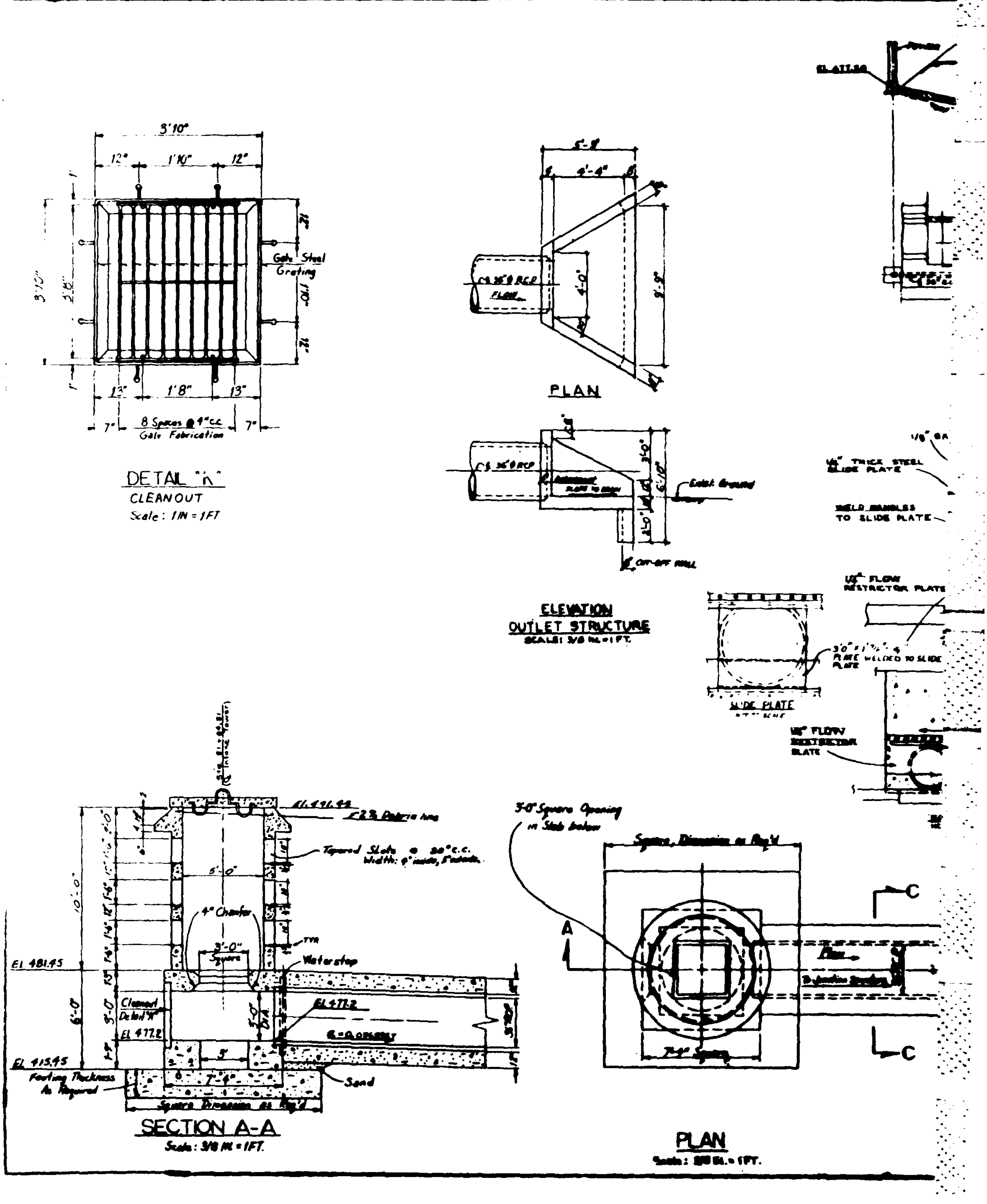
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN

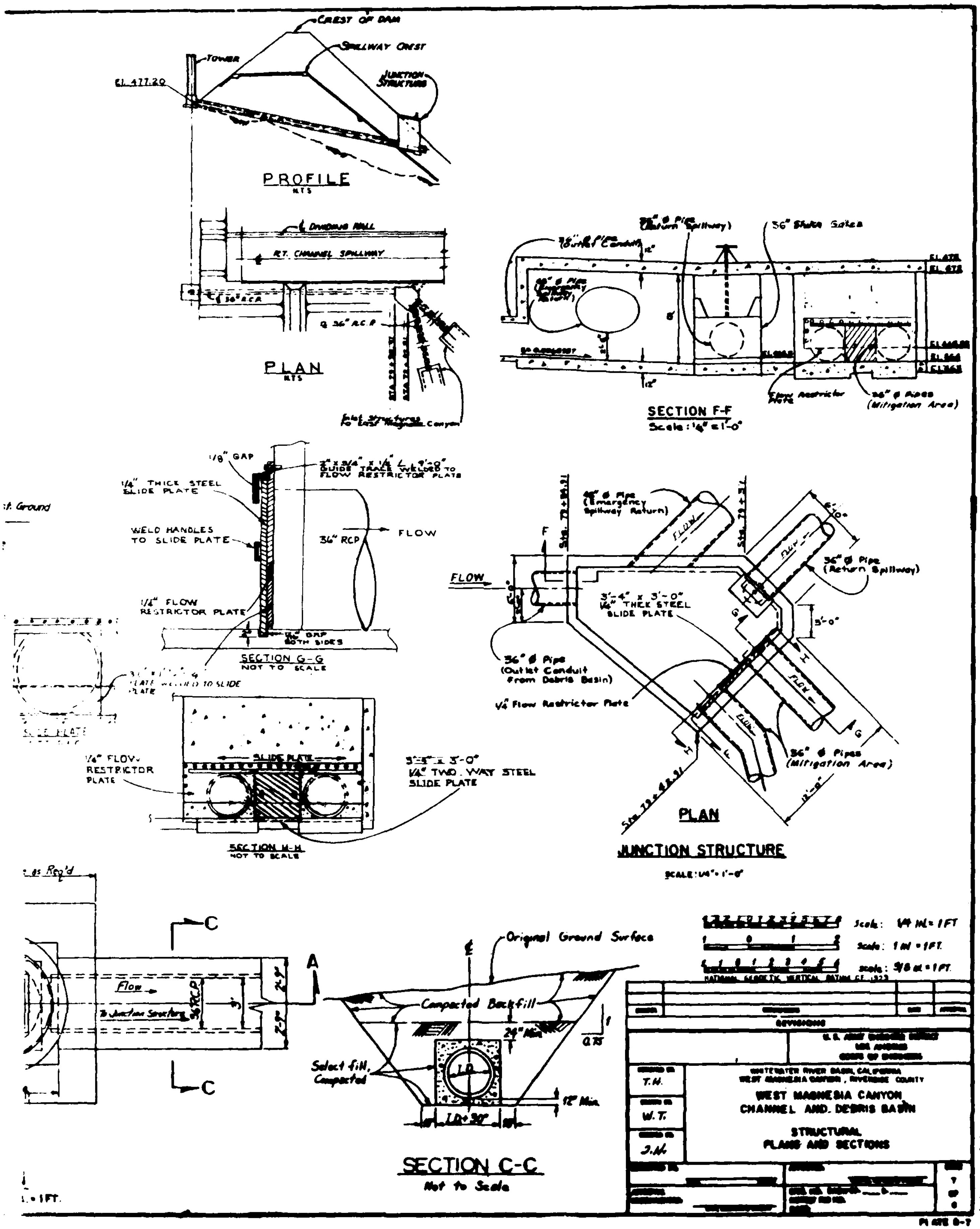
DEBRIS BASIN
PLAN AND SECTIONS

DESIGNED BY: M. S. / G. M. M.
DRAWN BY: H. Q.
CHECKED BY:
SUBMITTED BY:
DATE APPROVED
DISTRICT FILE NO

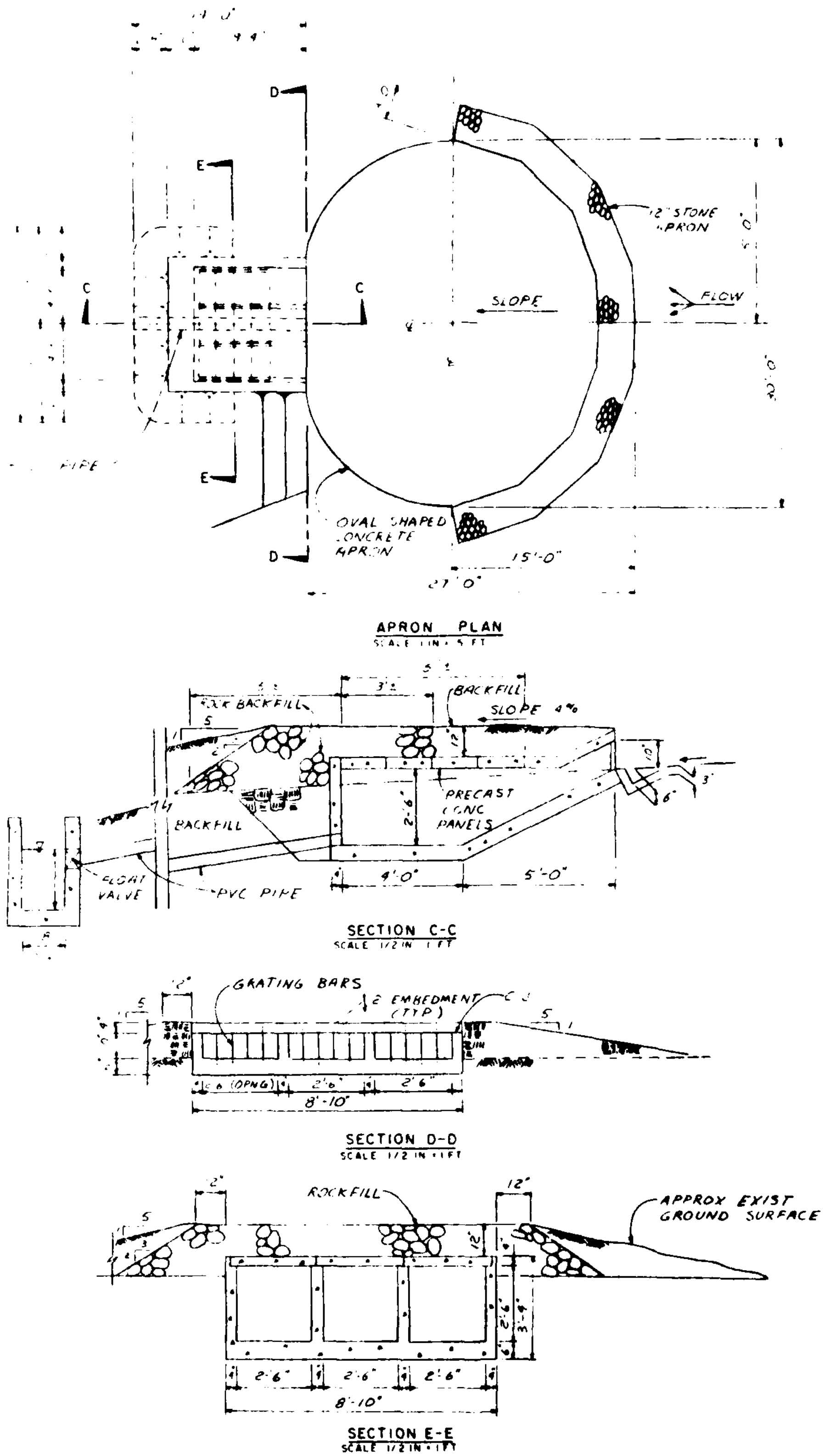
PLATE

II

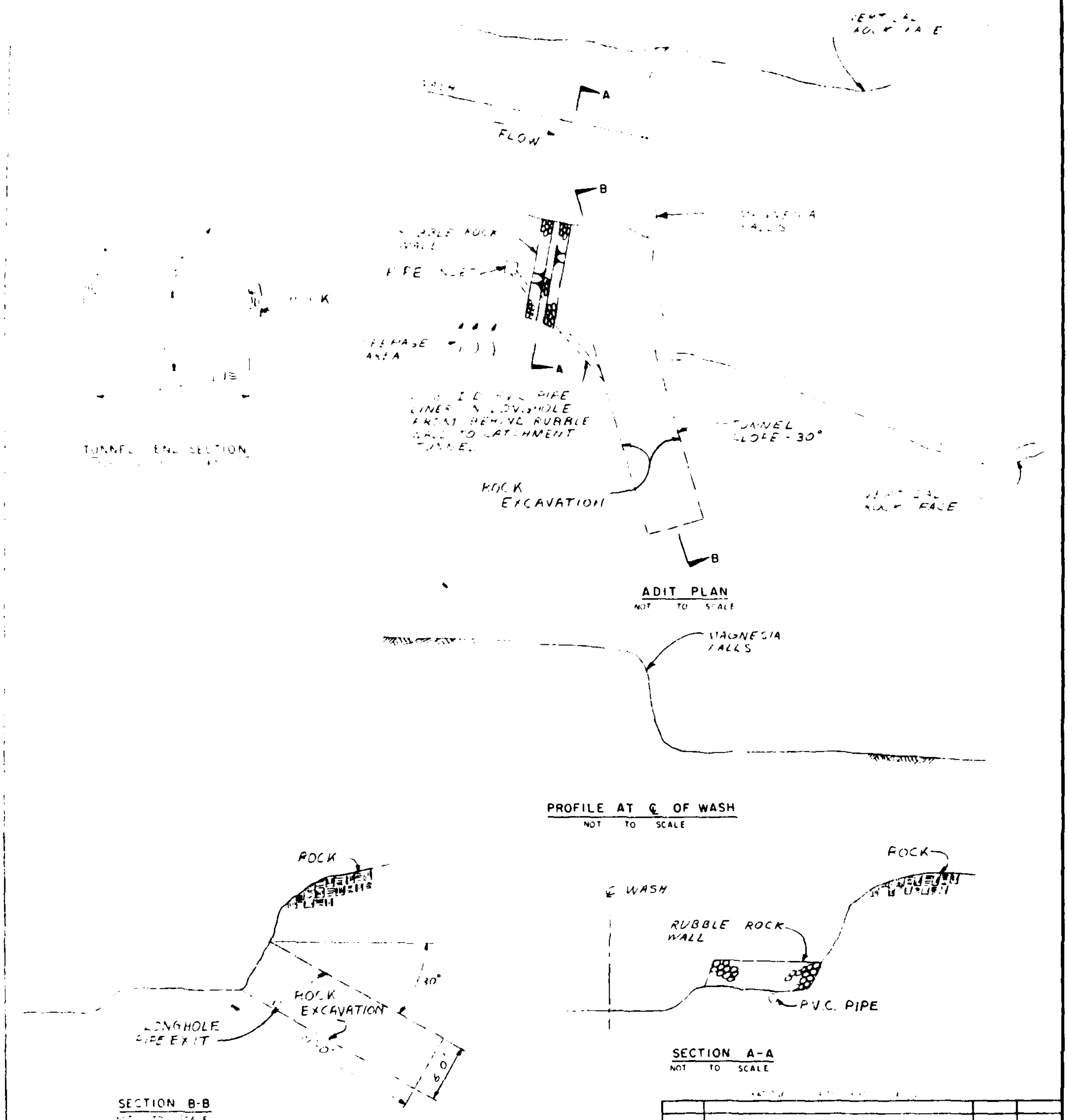

3'10"
12"
1'10"
12"
3'10"
3'8"
Galv Steel Grating
1'8"
8 Spaces @ 4" cc
Galv Fabrication
7"
DETAIL "A"
CLEANOUT
Scale: 1IN = 1FT
5'-8"
4'-4"
4'-0"
9'-9"
PLAN
3'-0"
6'-10"
Exist. Ground
CUT-OFF WALL
ELEVATION
OUTLET STRUCTURE
SCALE: 3/8 IN.=1FT.
1/2" THICK STEEL SLIDE PLATE
WELD HANDLES TO SLIDE PLATE
1/2" FLOW RESTRICTOR PLATE
PLATE WELDED TO SLIDE PLATE
SLIDE PLATE
1/2" FLOW RESTRICTOR PLATE
El 491.45
Tapered Slots @ 20" c.c.
5'-0"
4" Chamfer
3'-0" Square
Waterstop
El 481.45
10'-0"
6'-0"
Cleanout Detail "A"
El 477.2
El 475.45
Footing Thickness As Required
Sand
7'-4"
Square Dimension as Req'd
SECTION A-A
Scale: 3/8 IN. = 1FT.
3'-0" Square Opening in Slab below
Square Dimension as Req'd
A
C
7'-4" Square
PLAN



CREST OF DAM
SPILLWAY CREST
TOWER
JUNCTION STRUCTURE
EL 477.20
PROFILE
NTS
RT. CHANNEL SPILLWAY
PLAN
NTS
SECTION F-F
36" Slide Gates
36" Ø Pipes (Mitigation Area)
1/8" GAP
1/4" THICK STEEL SLIDE PLATE
WELD HANDLES TO SLIDE PLATE
36" RCP
FLOW
1/4" FLOW RESTRICTOR PLATE
SECTION G-G
NOT TO SCALE
1/4" FLOW RESTRICTOR PLATE
SLIDE PLATE
SECTION H-H
NOT TO SCALE
48" Ø Pipe (Emergency Spillway Return)
36" Ø Pipe (Return Spillway)
3'-4" x 3'-0" 1/4" THICK STEEL SLIDE PLATE
36" Ø Pipe (Outlet Conduit From Debris Basin)
1/4" Flow Restrictor Plate
36" Ø Pipes (Mitigation Area)
PLAN
JUNCTION STRUCTURE
Flow
To Junction Structure
36"RCP
Original Ground Surface
Compacted Backfill
Select fill, Compacted
12" Min.
SECTION C-C
Not to Scale
T.H.
W.T.
WEST MAGNESIA CANYON CHANNEL AND DEBRIS BASIN
STRUCTURAL PLANS AND SECTIONS



WILDLIFE GUZZLER



ADIT

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: S.D.B.
CHECKED BY:

WHITEWATER RIVER BASIN, CALIFORNIA
WEST MAGNESIA CANYON, RIVERSIDE COUNTY
WEST MAGNESIA CANYON
CHANNEL AND DEBRIS BASIN
WILDLIFE GUZZLER AND ADIT

SUBMITTED BY: DATE APPROVED: SPEC NO DACW09-____-____ SHEET 8 OF 8
DISTRICT FILE NO

APPENDIX E

MAGNESIA SPRING CANYON CHANNEL,

DETAILED PROJECT REPORT FOR FLOOD CONTROL

CITY OF RANCHO MIRAGE

RIVERSIDE COUNTY, CALIFORNIA

APPENDIX E

ECONOMICS

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

DECEMBER 1983

TABLE OF CONTENTS

TABLES

INTRODUCTION

This analysis estimates the benefits and costs of providing flood protection along West Magnesia Spring. Benefits result from flood damage reduction and land value enhancement. The economic benefits and costs are compared for three plans. Cost apportionment between Federal and non-Federal interests is discussed for the recommended plan.

METHODOLOGY

The estimates of the project costs and benefits for each alternative were based on October 1983 price levels. Each alternative was assumed to be operative for 100 years after construction. Sufficient allowance was made for annual operation and maintenance costs to insure the long-range functioning of each project. A 8-1/8 percent discount rate was then used to convert the construction costs into annual payments over the life of the project. Operation and maintenance costs were added to this to arrive at the total annual charges. Each alternative was designed to reduce flood damages and hazards. Flood damages prevented were calculated by comparing the damages expected to occur over the 100-year analysis period without a project with those damages expected to occur with a project in place.

PROJECT COST ESTIMATES

First Cost

The estimated financial first costs of the project ($7,829,000) include estimates for construction, engineering and design, supervision and administration, relocations, rights-of-way, beautification, mitigation, and allowance for contingencies. Unit prices were developed by using current October 1983 material, equipment, and labor costs for the basic facilities. To appraise the land costs, the sites of the recommended improvements were inspected and real estate markets concerned were analyzed. The cost of acquiring the rights-of-way was based on developments currently in place. Total first costs also include the non-financial cost of interest during construction (IDC) $208,000. This cost reflects 8-1/8 percent annual interest on construction costs only for an average of 4.5 months. It does not apply to lands and relocations because all lands would otherwise not be used for any purpose other than flood control easements. First cost including IDC total $8,037,000 for the preferred plan. Table E-1 shows the first costs of each alternative.

Annual Charges

Total first costs for the alternatives were converted to annual payments by applying the capital recovery factor at the current interest rate of 8-1/8 percent for 100 years. Estimated annual charges for operation and maintenance of the project were added to this annual payment. Annual charges thus include: (a) interest on total investment, (b) amortization of the total investment over the project life, and (c) average annual costs of project maintenance and operation. Table E-1 shows the annual charges computed for each project feature of each alternative.

PROJECT BENEFITS

Most alternative plans were formulated for a single use of the flood plain resources. Recognition was also given to such nonquantifiable beneficial impacts as the reduction in the threat to loss of life, the decrease in disease hazards, and savings in the cost of economic and social dislocation caused by large floods. They do not appear explicitly in the benefits estimates.

All alternatives provided two different types of flood control benefits; flood damage reduction (inundation benefits) and location benefits. The flood-damage-reduction category reflects the savings that can be attributed to the prevention of direct damages inflicted by floodwaters on real and personal property. Also included is some measure of the reduction of nonphysical losses that would be otherwise experienced by residents of the area in terms of lost wages and loss or return on capital investments. These flood-damage-reduction benefits were calculated by comparing the damages without any improvement with those damages that would occur if each alternative plan were in place. Such reductions during the project life (100 years for all alternatives) were claimed as a benefit. Location benefits accrue from permitting lands to be developed as a result of construction of a flood protection improvement.

Flood Damage Reduction

Flood-damage reduction benefits were estimated by evaluating damages that would occur to present (1983) and projected development if no project were constructed and then deducting the damages that would be expected to occur under the same conditions after the project was constructed. Damages are a function of type and value of damageable property, as well as hydrologic and topographic conditions.

Present Damageable Values

Present 1983 value of developments in the overflow area were obtained from many sources. Estimates of improvement values for private property were made by: (a) sampling development carried on the Riverside County Tax assessor's books and adjusting the assessed valuation to market value, (b) consulting knowledgeable real estate brokers for valuation data, and (c) performing field inspections and development appraisals using such references as the Marshall Valuation Service. The Los Angeles District conducted a survey of 18 insurance companies and claims adjusters in the District to determine the value of residential contents. Information was sought on home-owners fire insurance policies. These experts were asked specifically about the value of contents in houses that had been completely destroyed in or`er to exclude any smoke damage that might skew content damages. They reported that settlement for contents generally ranged from 40 to 60 percent. For better homes the rough estimate was 50 to 60 percent. Present values of damageable property is shown by land use and flood in Table E-2.

Future value of contents per residence was projected at the OBERS projected rate of increase in personal per-capita income (2.6 percent annually) for Riverside County. The value of contents was allowed to increase to a maximum of 75 percent of the value of the structure. No increase in value of other existing developments was claimed. A summary of estimated present and future value of damageable property in the 500-year SPF, 100-year, and 50-year overflow areas is presented in Table E-2.

Hydraulic Data

Hydraulic studies were made to determine the extent of the overflow area, the depth of inundation, and the velocity of flow for each major flood magnitude. Plate 3 in the main text outlines the assumed overflow area. Most of the structures in Rancho Mirage are built on pads at or near grade. The overflow area represents the probable path of flooding used in this economic analysis. The area subject to inundation displayed on plate 2 in the main text was used to compute location benefits only.

Depth-damage relationships were used to evaluate the impact of the anticipated flows on development in the flood plain. These relationships, which were developed for each land-use category from the local historical flood-damage reports, have been verified and adjusted for different hydrological conditions after each flood in the Los Angeles District. Depth-damage relationships for selected points are shown in Table E-3. These depth-damage relationships, when applied to damageable property, were used to develop flood damages.

Present land use is primarily single family residential development with an average density of approximately four units to an acre. Future development that may occur with the construction of our project is expected to have a density of 2 units to an acre. Table E-4 shows present land use by overflow area. Table E-5 shows damages under existing conditions by flood and land use.

Future Flood Damages Without Project

Damages for each type of land use were summed for each flood. These damages are displayed in tables E-6 through E-9. The damages expected to result from each size flood were weighted by the probability of occurrence of that flood by combining the damage-discharge and discharge-frequency curves. Standard damage-frequency integration techniques were then used to calculate average annual damages. Equivalent annual damages were computed next by summing the present worth of the expected annual damages and applying the capital recover factor (partial payment series) for a 8-1/8 percent discount rate. Probable and equivalent annual damages (8-1/8 percent, 100 years) are shown for the flood plain on table E-10.

Residual Damages

The impact of each alternative plan was evaluated by using the frequency curves associated with the improvements, with adjustments made for the new channel capacities. These curves were applied to the basic damage-discharge curves. Probable damages remaining with the project in place were calculated by integrating the "with project" frequency curves and the damage-discharge curves. Equivalent annual damages were calculated at a 8-1/8 percent discount rate for a 100-year project life. Probable and equivalent annual (8-1/8 percent, 100-year) damages remaining with the recommended plan are shown in table E-11.

Equivalent annual remaining damages also include induced damages of $14,000 for single family residential structures developed only with SPF protection and $7,000 to their contents. Induced damages to these land uses increase to $27,000 and $14,000 with 100-year protection. Table E-11 does not include induced damages in order for the table to be internally consistant with other tables displaying damages prevented (table E-12) and damages without the project (table E-10).

Flood Damages Prevented

Table E-12 displays damages prevented by the preferred plan. These benefits are the same for all other plans providing protection from the Standard Project Flood. Table E-13 displays damages prevented by 100-year protection.

Location Benefits

Location benefits accrue to this project by freeing approximately 150 acres for residential development. The acreage allowed to develop is presently prohibited from development by local ordinance because of the existing flood hazard. Location benefits equal the total increase in property value from a change in land use.

In the Coachella Valley, there is an extremely limited amount of land that offers the amenities of the Magnesia Canyon alluvial cone, which include freedom from the strong winds that blow throughout the Coachella Valley as well as desirable views of the valley floor.

The recommended plan is the most cost-effective solution for protecting the existing urban development on the West Magnesia Canyon alluvial cone. Incidental to the plan will also be flood protection to the vacant land on the cone. It is expected that the land will be developed for urban uses as a result of the project.

Location benefits are measured by the enhancement of property values. Increased market value has been estimated for the 150 acres of land affected by the project. The exact amount of increase is open to discussion owing to the fact that the risks potential developers are willing to take with this land are unknown. Instead, a range of possible current land values have been estimated. These values are then compared to $150,000 per acre with a project.

The range of possible property value increases implies the calculation of a range of possible values for location benefits which in turn produces a range of cost apportionment schedules. The minimum and maximum figures for land value, location benefits, and cost apportionment can be viewed as endpoints describing their respective ranges.

Location benefits range from $633,000 to $1,595,000 in equivalent annual terms. The minimum value is the annualized cost of constructing and maintaining a 100-year flood control plan. The rationale for using this figure is as follows. In order for a private sector individual to develop the land, he would have to provide 100-year flood protection for that land, as per FEMA requirements. In doing so, he would incur a cost of at least $633,000 in annualized terms.

The maximum location benefit value is simply the annualized value of the expected value of the 150 acres, minus the minimum value of land ($15,000 per acre from the Design Appendix). This value is calculated to be $1,595,000. Since approximately 150 acres in the Upper Cone will become developable under the project, and the area is primarily controlled by a single party, special location benefits are indicated and special cost apportionment required.

The important point of this discussion is the impact location benefits have on cost apportionment, project justification, and plan selection. All location benefit values within the calculated range lead to Federal costs apportionment in excess of $4 million. However, since Federal participation in this project is limited to $4,000,000 under the Small Project Authority, cost sharing for the Federal portion is unaffected by any value for location benefits in the calculated range. All location benefit values within the calculated range would adjust the net benefits of each plan equally. The only significant change would be justification of Plan 4 (Earthfill Dam) with use of equivalent annual location benefits exceeding $1,133,000.

Maximization of Net Benefits

Net benefits are maximized at the level of protection where benefits exceed costs by the greatest margin. Table E-1 lists net benefits for each plan considered. Plan 1A maximizes net NED benefits with $396,000 net equivalent annual benefits for 100-year level of protection. This qualifies Plan 1A as the NED plan. Plan 1, the preferred plan provides additional flood protection to the SPF level. In equivalent annual terms, this extra protection costs $128,000 for $51,000 in NED benefits, for a net loss of $77,000. The rationale for recommending Plan 1 instead of Plan 1A is explained in the main text.

COST APPORTIONMENT

Sharing of cost between Federal and non-Federal interest for the recommended plan is based on Federal legislation pertaining to local protection projects and administrative determinations. Under present policy, local interests are required to provide necessary rights-of-way for the project, relocate all highways, utilities, and irrigation and drainage facilities, and maintain and operate all works following completion. In addition, local participation in

the construction costs is required when large land value appreciation of special local benefit occurs as a result of the project. Analysis had indicated that the location benefits should be classified as special local benefits. Local participation in construction cost is thus required. This participation amounts to: 50 percent x (percentage of benefits attributed to windfall benefits x project costs) - 100 percent x (percentage of benefits attributed to windfall benefits x lands and relocation costs).

The non-windfall portion of benefits, damages prevented, is reduced by the amount of induced damages. The resultant figure, net damages prevented, is used to determine cost sharing.

Only financial costs, actual expenditures, are cost shared. Interest during construction is not cost shared because it is not an actual expenditure.

Where the amount of local interest as described above is not enough to pay for the amount of first costs in excess of the Federal limitation, local interests would be required to provide additional monies so that the Federal share does not exceed the limitation. Under the Small Project Authority, Federal share of construction is limited to $4 million.

Contribution towards first cost required from local interests are given in tables E-14 and E-15. As can be seen, the local share of construction costs are $2,267,500 as a result of special local benefits. Cost of lands and relocations equals $388,000. Together, the local first cost share is $2,655,500. Since the amount of construction cost in excess of the $4 million Federal limitation is estimated at $4,217,000, local interests would be required to contribute this larger amount instead of $2,655,500. The apportionment is thus as follows: Federal government, $4,000,000; local interests $4,217,000 of which $388,000 is for lands and relocations and $3,829,000 is for construction costs in excess of $4,000,000.

TABLE E-1

ECONOMIC SUMMARY
(8-1/8% 100 Year Project Life)
(1983 $1000)

FLOOD CONTROL	Plan 1 Debris Basin and Rectangular Channel (SPF)	Plan 1A Debris Basin and Rectangular Channel (100 yr)	Plan 4 Earthfill Dam (SPF)
First Cost			
Construction	7,441	6,062	16,033
Interest During Construction (IDC)	208	171	1,290
Right of Way	388	388	388
Total	8,037	6,621	17,711
ANNUAL CHARGES			
Construction with IDC and Rights of way	653	538	1,440
Operation and Maintenance	72	59	72
Total	725	597	1,512
ANNUAL BENEFITS			
Flood Damages Prevented	432	401	432
Induced Damages	(21)	(41)	(21)
Location	633	633	633
Total Benefits	1,044	993	1,044
NET BENEFITS	319	364	-468
B/C	1.44 (1.4)	1.6 (1.7)	.69 (.7)

TABLE E-3

Inside Depth-Damage Relationship
(In Percent)

LAND USE:	Water Depth in Feet 0	.5	1	1.5	2	2.5
Residential						
Single Family - Structures	0	4	7.8	11.6	16.9	21.8
- Contents	0	5	7	10	15	20
Multi-Family - Structures	0	3	6	9	13	16
- Contents	0	5	7	10	15	20
Commercial						
Retail - Structures	0	3	6	9	13	16
- Contents	0	5	7	10	15	20
Office - Structures	0	7.5	12.5	20	26.5	33
- Contents	0	5.5	12.5	21.5	33	44
Food Markets - Structures	0	1	5	10	16	23
- Contents	0	20	50	60	80	100
Public Office - Structures	0	7.5	12.5	20	26.5	33
- Contents	0	5.5	12.5	21.5	33	44

TABLE E-4

Land Use in Overflow Area
Number of Units

	Without Project				With Project		
	Present (1983)	PY1 (1984)	1994	2004-2084	PY1 (1984)	(1994)	2004-2084
Residential							
Single Family	268	274	284	286	274	587	589
Multi Family	212	262	352	372	272	407	432
Commercial							
Retail	56	56	56	56	56	56	56
Office	16	32	50	54	28	52	56
Food Markets	2	2	2	2	2	2	2
Public Offices	1	1	1	1	1	1	1
Total	555	627	745	771	633	1,105	1,136

TABLE E-5

DAMAGES UNDER EXISTING (1983) CONDITIONS BY FLOOD & LAND USE
(1983 $1000)

	500 yr	SPF	100 Yr	50 Yr
Residential				
Single Family Structures	$3,610	$2,980	$1,910	$1,290
Single Family Content	1,380	1,040	690	490
Multi Family Structures	900	730	510	340
Multi Family Contents	480	360	230	170
Commercial				
Retail	3,220	2,560	1,760	1,190
Office	1,200	620	690	420
Food Markets	800	640	470	360
Public	90	80	60	30
Utilities & Roads	40	20	20	20
Flood Control	120	60	40	30
Total	11,800	9,090	6,380	4,340

TABLE E-6

TOTAL DAMAGES
BY PROPERTY TYPE
(1983 $1000)

500 YEAR FLOOD

	1982	1984	1994	2004	2014	2020-84
Residental						
Single Family Structures	$3,610	$3,710	$3,880	$3,910	$3,910	$3,910
Single Family Contents	1,380	1,580	2,090	2,690	2,690	2,690
Multi-Family Structures	900	1,220	1,900	1,900	1,900	1,900
Multi-Family Contents	480	690	1,390	1,900	1,900	1,900
Commerical						
Retail	3,220	3,220	3,220	3,220	3,220	3,220
Office	1,200	4,520	8,260	9,090	9,090	9,090
Food Market	770	770	770	770	770	770
Public Offices	90	90	90	90	90	90
Utilities and Roads	40	40	40	40	40	40
Flood Control Structures	120	120	120	120	120	120
TOTAL	11,810	15,960	21,760	23,730	23,730	23,730

TABLE E-7

SPF DAMAGES
BY PROPERTY TYPE
(1983 $1000)

	SPF					
	1983	1984	1994	2004	2014	2024-84
Residential						
Single Family Structures	$2,800	2,870	3,010	3,040	3,040	3,040
Single Family Contents	1,040	1,150	1,560	2,020	2,020	2,020
Multi-Family Structures	730	1,000	1,650	1,650	1,650	1,650
Multi-Family Contents	360	520	1,040	1,410	1,430	1,430
Commercial						
Retail	2,560	2,560	2,560	2,560	2,560	2,560
Office	940	3,570	6,520	7,180	7,180	7,180
Food Markets	640	640	640	640	640	640
Public Offices	80	80	80	80	80	80
Utilities and Roads	20	20	20	20	20	20
Flood Control Structures	60	60	60	60	60	60
Total	9,230	12,470	17,140	18,660	18,660	18,660

TABLE E-8

100 YEAR FLOOD DAMAGES
BY PROPERTY TYPE
(1983 $1000)

	1983	1984	1994	2004	1014	2024-84
Residential						
Single Family Structures	$1,910	1,980	2,060	2,090	2,090	2,090
Single Family Contents	690	780	1,040	1,350	1,350	1,350
Multi-Family Structures	510	690	1,070	1,140	1,140	1,140
Multi-Family Contents	230	350	690	950	950	950
Commercial						
Retail	1,760	1,760	1,760	1,760	1,760	1,760
Office	690	2,600	4,760	5,240	5,240	5,240
Food Markets	470	470	470	470	470	470
Public Offices	60	60	60	60	60	60
Utilities	20	20	20	20	20	20
Flood Control Structures	40	40	40	40	40	40
Total	6,380	8,750	11,970	13,120	13,120	13,120

TABLE E-9

50 YEAR FLOOD DAMAGES
By Property Type
(1983 $1000)

	1983	1984	1994	2004	2014	2020-84
Residential						
Single Family Structures	$1,290	1,330	1,390	1,400	1,400	1,400
Single Family Contents	490	540	730	950	950	950
Multi-Family Structures	340	460	710	760	760	760
Multi-Family Contents	170	240	490	660	670	670
Commercial						
Retail	1,190	1,190	1,190	1,190	1,190	1,190
Office	420	1,600	2,930	3,230	3,230	3,230
Food Markets	360	360	360	360	360	360
Public Offices	30	30	30	30	30	30
Utilities	20	20	20	20	20	20
Flood Control Structures	30	30	30	30	30	30
Total	4,340	5,800	7,880	8,630	8,640	8,640

TABLE E-10

Probable and Equivalent Annual Damages Without Project
(1983, $1000)

	1983	1984	1994	2004	2014	2020-24	Equivalent Annual 100 Yr, 8-1/8%
Residential							
Single Family Structures	82	84	88	88	88	88	86
Single Family Contents	31	34	47	60	60	60	46
Multi-Family Structures	22	30	46	48	48	48	42
Multi-Family Contents	11	16	31	41	41	41	30
Commercial							
Retail	[illegible]	74	74	74	74	74	74
Office	28	104	189	207	207	207	173
Food Markets	21	21	21	21	21	21	21
Public Offices	2	2	2	2	2	2	2
Utilities	1	1	1	1	1	1	1
Flood Control	3	3	3	3	3	3	3
Total	275	369	502	545	545	545	478

TABLE E-11

Probable and Equivalent Annual Residual Damages
(1983 $1000)

SPF CHANNEL or SPF DAM

	1983	1984	1994	2004	2014	2020-84	Equivalent Annual 100 Yr, 8-1/8%
Residential							
Single Family Structures	9	9	9	9	9	9	9
Single Family Contents	3	3	5	6	6	6	4
Multi-Family Structures	2	2	5	5	5	5	4
Multi-Family Contents	1	1	3	3	3	3	3
Commercial							
Retail	7	7	7	7	7	7	7
Office	2	11	20	21	21	21	18
Food Markets	2	2	2	2	2	2	2
Public Offices	0	0	0	0	0	0	0
Utilities and Roads	0	0	0	0	0	0	0
Flood Control Structures	0	0	0	0	0	0	0
Total	26	35	51	53	53	53	47

Probable and Equivalent Annual Damages Prevented With Project
(1983 $1000)

	SPF CHANNEL or SPF DAM						
	1983	1984	1994	2004	2014	2020-84	Equivalent Annual 100 Yr, 8-1/8
Residential							
Single Family Structures	73	75	79	79	79	79	78
Single Family Contents	28	31	42	54	54	54	42
Multi-Family Structures	20	28	41	43	43	43	37
Multi-Family Contents	10	15	28	38	38	38	29
Commercial							
Retail	67	67	67	67	67	67	67
Office	26	93	169	186	186	186	154
Food Markets	19	19	19	19	19	19	19
Public Offices	2	2	2	2	2	2	2
Utilities and Roads	1	1	1	1	1	1	1
Flood Control Structures	3	3	3	3	3	3	3
Total	249	334	451	492	492	492	432

TABLE E-13

Probable and Equivalent Annual Damages Prevented With Project
(1983 $1000)

100 YEAR CHANNEL

	1983	1984	1994	2004	2014	2020-84	Equivalent Annual 100 Yr. 8-1/8%
Residential							
Single Family Structures	66	67	71	71	71	71	70
Single Family Contents	24	27	36	48	48	48	38
Multi-Family Structures	17	23	35	38	38	38	33
Multi-Family Contents	9	12	24	33	34	34	24
Commercial							
Retail	61	61	61	61	61	61	61
Office	21	72	151	166	166	166	153
Food Markets	17	17	17	17	17	17	17
Public Offices	1	1	1	1	1	1	1
Utilities and Roads	1	1	1	1	1	1	1
Flood Control Structures	3	3	3	3	3	3	3
Total	220	284	400	437	438	438	401

TABLE E-14

CONTRIBUTION OF NON-FEDERAL INTERESTS

Flood Control Financial First Costs

Construction	7,829,000
Lands and Relocations	388,000
Total	8,217,000

Flood Control Benefits

	Non-windfall portion	Windfall portion	Total
Net Damages Prevented*	$411,000	0	411,000
Location	0	633,000	633,000
Total	411,000	633,000	1,044,000
Percent	39	61	100

Apportionment of Flood Control Cost

	Non-windfall portion	Windfall portion	Total
Construction	3,057,000	4,772,000	7,829,000
Land Relocations	151,000	237,000	388,000
Total	3,208,000	5,009,000	8,217,000

* Net Damages Prevented is damages prevented less induced damages.

TABLE E-15

TENTATIVE APPORTIONMENT

	Federal	Non-Federal	Adjusment of share to assign all land and relocations to non-federal interest	
			Federal	Non-Federal
Non-Windfall				
Construction	3,057,000	0	3,057,000	0
Lands and Relocations	0	151,000	0	151,000
Total	3,057,000	151,000	3,057,000	151,000
Windfall				
Construction	2,386,000	2,386,000	2,504,500	2,267,500
Lands and Relocations	118,500	118,500	0	237,000
Total	2,540,500	2,504,500	2,504,500	2,504,500
Total Flood Control First Costs				
Construction	5,443,000	2,386,000	5,561,500	2,267,500
Lands and Relocations	118,500	269,500	0	388,000
Total	5,561,500	2,655,500	5,561,500	2,655,500

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER	2. GOVT ACCESSION NO. AD-A150	3. RECIPIENT'S CATALOG NUMBER 305
4. TITLE (and Subtitle) West Magnesia Canyon Channel City of Rancho Mirage Riverside County, California		5. TYPE OF REPORT & PERIOD COVERED Detailed Project Report Final Dec 1983-
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s)		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS		12. REPORT DATE December 1983
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		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distrubution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

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18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Flood Contol Planning

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

West Magnesia Spring Canyon Creek, which originates in the Santa Rosa mountains, skirts the western side of Rancho Mirage along the foothills as it leaves the canyon and flows a distance of about 1.5 miles before it empties into the Whitewater River. The drainage basin comprises about 5 square miles.

The main objectives of this study are to provide a high degree of flood protection to the residents of Rancho Mirage and to protect the Nation's

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environment.

The Corps recommends that, subject to certain conditions of nonFederal cooperation as outlined in this report, the proposal for flood control be approved for construction. The total cost of the recommended plan is estimated at $8,279,000. The Federal share of the estimated cost would be $4,000,000, and the non-Federal share would be $4,279,000 of which $3,891,000 is for construction and $388,000 is for lands, easements, rights-of-way and relocations.

The local sponsor of the project is the Coachella Valley Water District.

END

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